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TAJUK PROJEK : LIME STABILIZATION OF MARGINAL SUB-GRADE AS CAPPING LAYER FOR ROAD CONSTRUCTION

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LIME STABILIZATION FOR MARGINAL SUB-GRADE AS CAPPING LAYER FOR ROAD CONSTRUCTION

(PENSTABILAN KAPUR TERHADAP SUB-GRED LEMAH SEBAGAI LAPISAN PENUKUP UNTUK PEMBINAAN JALAN)

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ABSTRACT

Cohesive soils, which contain high percentage of moisture have low values of California Bearing Ratio (CBR) and shear strength. Standard Specification for Road Works published by the Malaysian Public Works Department (JKR) suggests that these weak soils should be replaced with a suitable capping layer. However, instead of replacing the weak soil layer, stabilization by lime can also be carried out on cohesive soils. In this research, lime stabilization method was applied at the minimum strength of the soil to form capping layer. Cohesive soils with Plasticity Index (PI) of more than 20 % and Clay Fraction (CF) exceeding 30 % were used. Hydrated lime in the percentage of 5 % by mixture was used as the stabilizing agent. CBR test were conducted on the stabilized soil samples, and the results show strength development of more than 100 % after a curing period of 28 days. This strength development in lime-stabilized soil was also observed experimentally by load testing of a road structure model which consists of lime-stabilized layer as the capping layer over a weak, clay sub-grade. Initially, observations are made on the ability of the road structure to reduce settlement of the sub-grade when load is applied. Unstabilized soil experienced over 100 % settlement when the load increased from 0.5 kN to 1.5 kN. A reduction in settlement from 70% to 100% was achieved for stabilized layer immediately after mixing. After 28 days, the settlement is almost negligible. The soil bearing capacity of stiff layer overlaying soft layer was determined using the theory proposed by Braja (1999). The theory is confirmed by the bearing capacity model study based on stabilized layer overlay soft soil conducted using 32 cm x 32 cm model box with 52 cm in height. A number of 26 samples were prepared with 3 different stabilized layers i.e., 40, 60 and 80 mm thick. These samples were cured for a period of 0, 7, 14 and 28 days. The bearing capacity was determined according to the theoretical analysis and from model testing. The bearing capacity increases with curing period and thickness of the capping layer. For the purpose of weight limit design, curing period of 28 days is considered. Weight limit design with the incorporation of geotechnical aspect is used to determine the maximum allowable vehicular load. This maximum allowable load is taken at a point before the ultimate bearing capacity of the sub-grade is exceeded. This approach of design provide an additional precaution against accessive vehicular load so as not to overstress the sub-grade.

ABSTRAK

Tanah berjelekit yang mengandungi air yang tinggi mempunyai nilai Nisbah Galas California (CBR) dan kekuatan ricih yang rendah. Untuk tujuan pembinaan sub-gred jalan, Spesifikasi Pembinaan Jalan oleh Jabatan Kerja Raya (JKR) menyarankan agar tanah seumpama ini digantikan dengan satu lapisan penukup yang sesuai. Selain daripada proses penggantian bahan, lapisan penukup ini juga boleh disediakan dengan menggunakan kaedah penstabilan kapur terhadap tanah berjelekit. Di dalam penyelidikan ini, kaedah penstabilan kapur diaplikasikan pada kekuatan tanah yang minimum untuk pembinaan lapisan penukup. Tanah berjelekit yang mempunyai indeks keplastikan (PI) lebih daripada 20 % dan pecahan tanah halus (CF) lebih 30 % telah digunakan. Kapur terhidrat sebanyak 5 % telah dicampurkan sebagai agen penstabilan. Ujikaji CBR dijalankan terhadap tanah terstabil kapur dan berdasarkan keputusan ujikaji, ia menunjukkan pertambahan kekuatan melebihi 100 % selepas tempoh awetan 28 hari. Peningkatan kekuatan tanah terstabil kapur juga dilihat melalui ujikaji terhadap model yang menggambarkan pembinaan struktur lapisan penukup terstabil kapur di atas lapisan tanah liat yang lembut. Pada awalnya, pemerhatian dibuat terhadap keupayaan struktur untuk mengurangkan mendapan pada sub-gred apabila dikenakan beban. Tanah tak terstabil menunjukkan peningkatan mendapan lebih daripada 100% dengan kenaikan beban dari 0.5 kN hingga 1.5 kN. Mendapan dapat dikurangkan daripada 70% hingga 100% untuk tanah terstabil kapur setelah proses penstabilan dijalankan. Selepas 28 hari, kesan mendapan boleh diabaikan. Keupayaan galas untuk lapisan tanah keras di atas lapisan tanah lembut ditentukan menggunakan teori yang dicadangkan oleh Braja (1999). Teori tersebut telah dipastikan berdasarkan kajian keupayaan galas tanah terstabil kapur di atas tanah liat lembut dengan menggunakan kotak model berukuran 32 cm x 32 cm dan 52 cm tinggi. Sebanyak 26 sampel model disediakan dengan ketebalan lapisan terstabil kapur pada 40, 60 dan 80 mm. Sampel ini diawet untuk tempoh 0, 7, 14 dan 28 hari. Keupayaan galas struktur berdasarkan ujikaji model dan teori ditentukan dan ia menunjukkan peningkatan mengikut tempoh awetan dan tebal lapisan terstabil kapur. Pada tempoh awetan 28 hari, keupayaan galas struktur diambil kira sebagai ciri untuk tujuan rekabentuk had berat. Rekabentuk had berat dengan aspek geoteknik diperkenalkan untuk menentukan beban maksimum daripada kenderaan. Beban maksimum ini diambil kira agar ia tidak melebihi keupayaan galas muktamad sub-gred. Kaedah rekabentuk ini dapat memberikan langkah dalam menentukan beban daripada kenderaan agar keupayaan galas sub-gred tidak dilampaui.

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LIST OF ABBREVIATIONS

μ	-	Poisson ratio
γ_b	-	Bulk density
°C	-	Degree celcius
γ_d	-	Dry density
°F	-	Degree farenheit
μm	-	micrometer
ΔP	-	Deflection within the pavement layer = 0
σ_r	-	Radial stress
ΔS	-	Deflection within the sub-grade
ΔT	-	Total surface deflection
Φ_u	-	Frictional component
σ_z	-	Vertical normal stress
A, B, C, F and H	-	Function values which depend on z/a and r/a
ABC	-	Aggregate Base Course
AC	-	Asphaltic concrete
A_c	-	Clay activity
Al_2O_3	-	Alumina
ALC	-	Available lime content
ASTM	-	American Society for Testing and Materials
B	-	Width of the foundation
B_{Max}	-	Maximum width of the contact area of the stress from surface
B_{Min}	-	Minimum width of the contact area of the stress from surface

BSI	-	British Standard Institution
C_1	-	Undrained shear strength of the lime-stabilized layer
C_2	-	Undrained shear strength of the soft clay layer
C_a	-	Adhesion force
$Ca(OH)_2$	-	Calcium hydroxide
$CaCO_3$	-	Calcium carbonate
CAH	-	Calcium Aluminate Hydrate
CaO	-	Calcium oxide
CBR	-	California Bearing Ratio
CH	-	High plasticity
CO_2	-	Carbon dioxide
CSH	-	Calcium Silicate Hydrate
C_u	-	Cohesive component
DT_p	-	Department of Transport
E	-	Modulus Young
ESA	-	Equivalent Standard Axle
FS	-	Factor of safety
g	-	Gram
H_2O	-	Water
HMAC	-	Hot Mix Asphalt Concrete
H_R	-	Road thickness
H_s	-	Thickness of lime-stabilized layer
ICL	-	Initial Consumption of Lime
kg/m^3	-	kilogram per meter cube
kN	-	Kilo Newton
kPa	-	kilo pascal
L	-	Length of the foundation
l	-	Litre
LL	-	Liquid Limit
LOI	-	Loss on ignition
LSS	-	Lime Stabilized Sub-grade

LVDT	-	Linear Vertical Displacement Transducer
m^3	-	Metre cube
MDD	-	Maximum Dry Density
Mg	-	Mega gram
$Mg(OH)_2$	-	Magnesium hydroxide
MH	-	High plasticity silt
mL	-	Mililitre
mm	-	millimeter
OMC	-	Optimum moisture content
P	-	Point load
PCC	-	Portland cement Concrete
pH	-	pH value
PI	-	Plasticity Index
q_t	-	Bearing capacity of the stiff layer
q_u	-	Ultimate bearing capacity
r	-	Radial distance from point load
RI	-	Reactivity index
SiO_2	-	Silica
$T_{A'}$	-	Corrected equivalent thickness
TRRL	-	Transport and Road Research Laboratory
UCS	-	Unconfined Compressive Strength
UK	-	United Kingdom
USA	-	United States of America
USCS	-	Unified Soil Classification System
V	-	The titration volume in mL
z	-	Depth of the point from the surface

LIST OF APPENDICES

CHAPTER 1

INTRODUCTION

1.1 Introduction

Lime stabilization has been adopted for road building in European countries, Africa and particularly the United State of America (USA) for many years. The use of lime stabilization in the United Kingdom (UK) has received an impetus in recent years from its incorporation into the Department of Transport (DT_p) Specification for Highway Works to form capping for road pavement although it has been used in several other applications such as parking areas, airfield pavements, temporary access roads, surfacing and modifying bulk fill (Barnes and Reynolds, 1989). As for road construction industry in Malaysia, much more basic research on lime stabilization with respect to Malaysian soils need to be done in order to guarantee applications and effectiveness of lime stabilization for road construction in the local context.

1.2 Background of the Problem

In the construction works, it is especially important to remember that without a properly prepared sub-grade, it is difficult, if not impossible, to construct a satisfactory sub-base and base (Metcalf, 1977). The sub-grades also must be able to sustain traffic loading without excessive deformation and this is controlled by the vertical compressive stress or strain at the formation level. As shown in Figure 1.1, it is one of the examples of the road construction through the marginal soil. On weak sub-grades, it is common practice to use a capping layer between the sub-grade and the sub-base to provide a suitably firm surface for the placement and compaction of the sub-base. Figure 1.2 shows the capping layer was constructed before the placement of the sub-base. The material for the capping layer is basically granular material and imported from other sites. However, this is not an economical and environmental option, because it will decrease the sources of the granular materials. Sherwood (1993) mentioned that when the California Bearing Ratio (CBR) of the sub-grade is less than 15 %, British design criteria require the use of a suitable capping layer of a low-cost local material. The capping layer can be constructed from granular material to a lower specification than the sub-base itself but there are clear advantages of stabilizing the top of the sub-grade to form the capping layer. The in-situ stabilization of the sub-grade with lime may offer economic and environmental advantages. The function of the capping layer is to protect the sub-grades from the adverse effects of wet weather. It provides a working platform on which sub-base construction can proceed with minimum interruption during wet weather and to allow the full load-spreading capabilities of the sub-base to be realized, which would not be possible were it to be laid directly on top of a weak sub-grade.

The common practice in designing pavement with stabilized sub-grade is the consideration of the bearing capacity of the structure (both stabilized and unstabilized soil). The stabilized structure gives the improvement in terms of reduced road thickness and the use of granular materials. However, the stabilized sub-grade should be protected from being overstressed to avoid failure. The wheel load is transmitted to the pavement through the tyre and the road structure distributes the wheel load to certain stress on the sub-grade. Therefore, the maximum load of the vehicles, which not overstressed the sub-grade is required. Based on the bearing capacity of the lime-stabilized capping layer underlain soft clay layer, factor of safety and materials for the road structure, the maximum load from the vehicle accepted for the certain thickness of the road was determined. This research will impose the weight limit of the road, which constructed on the lime-stabilized capping layer underlain soft clay layer. In highway engineering concept, the parameters considered in the designing of pavement include traffic volume which incorporates annual growth, strength of the foundation and the combination of the material to be used for a given service period of the road.

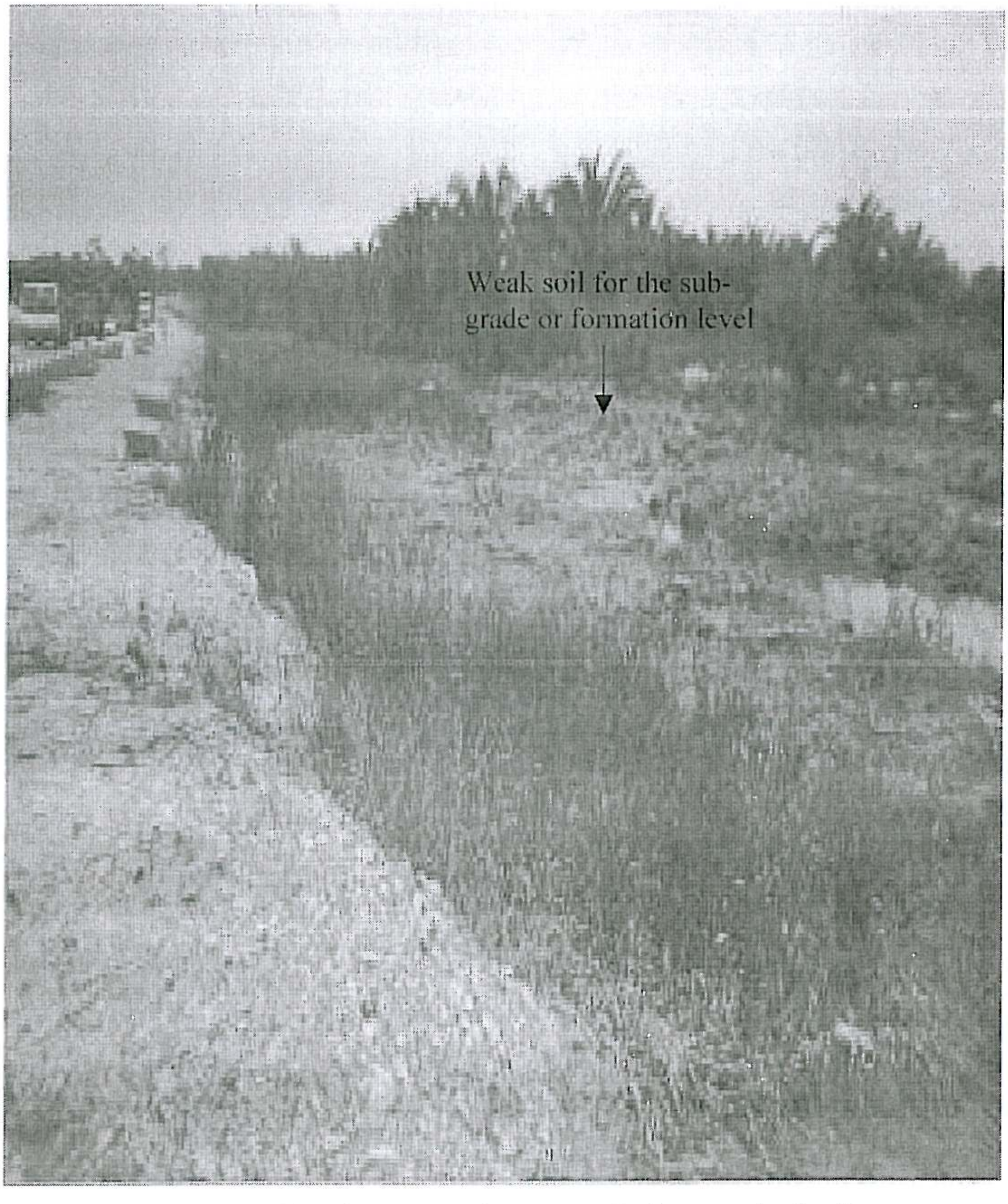


Figure 1.1: Road construction through the marginal soils

Figure 1.2: The capping layer was constructed before the placement and compaction of the sub-base

1.3 Research Objectives

The main objective of the research is to study the applicability of lime stabilization for marginal sub-grade as a capping layer for road construction. The specific objectives of the research are:

a. To establish soil-lime mixture design for capping layer in road construction.

b. To incorporate geotechnical aspect in road design by introducing the concept of stiff layer overlaying soft layer for the sub-grade.

1.4 Research Scope

The scopes of the research are:

a. Cohesive soils of various plasticity (ranging from 20 to 50 per cent) and clay fractions are used.

b. Only hydrated lime is used as a stabilizing agent.

c. The percentage of lime used is based on the Initial Consumption of Lime (ICL) test results and an additional of 2 % for the stabilization process.

d. Preparation of the soil-lime mixtures will be based on the dry mixing method and the moisture content is based on at least 95 % of the Maximum Dry Density (MDD) with a mellowing period of at least an hour.

e. The soil-lime mixtures were cured at different ages from 1 to 56 days under room temperature of 27 ± 2 °C and soaked in water at temperature of 20 ± 2 °C for the normal soaking period of 4 days prior to the CBR test. This is to establish the effect of curing duration and mixture strength.

f. The percentage of moisture in the preparation of the slurry samples to represent soft layer was based on the Liquid Limit (LL).

g. The slurry samples will be consolidated with the pressure of 5 kPa for 7 days before the construction of lime stabilized capping layer.

h. Only static load is applied to investigate the strength of lime stabilized capping layer over soft clay.

1.5 Significance of the Research

There is a growing recognition that stabilization is often the most cost-effective treatment when weak, marginal and moisture-susceptible clay soils are encountered. Significant savings can be achieved, and for several reasons, there is virtually no need to remove unwanted material from the site, or to import large quantities of bulk fill. This definitely cuts down on material transportation charges. With lime stabilization, site preparation can be carried out quickly and this in turn helps to speed up the whole project.

Nowadays, contractors have to take environmental effects into consideration. Stabilization with lime means far less tipping of spoil and far less demand on existing stone reserves or new quarries. Perhaps most importantly, it dramatically cuts the heavy traffic to and from the site which relieves an often intolerable burden on the local community and on the road system.

From Road Note 29 and 31, the road thickness design was based on the empirical approach. This research is to design weight limit of the vehicles for the road which not overstressed the lime-stabilized layer underlain soft clay layer sub-grade. Beside that, the design also can be a supplement to the conventional method, which is based on the number of axles from the traffic and the strength of the sub-grade. The proposed weight limit design is based on geotechnical theory even though it does not represent the actual situation of loading system. The theory is supported by the model study, which represents a stabilized layer overlying a soft layer.

CHAPTER II

LITERATURE REVIEW

Lime is one of the oldest and most versatile and vital of chemicals available to the engineer. There is a growing recognition that stabilization using lime is often the most cost effective treatment when weak, marginal and moisture-susceptible clay soils are encountered. Sub-grade stabilization includes stabilizing fine-grained soils in place (sub-grade) or borrowing materials which are employed as sub-base, such as hydraulic clay fills or otherwise poor quality clay and silty material obtained from cuts or borrow pits. The stabilized in-place sub-grade is suitable for support of an overlying base course or an asphalt surface layer. On the other hand, for most cases, the enhanced engineering properties obtained as a result of lime stabilization can and should be included, or at least considered, in the pavement design and in the design of subsequent overlaying courses.

2.1 Introduction

From the experimental work done by Rogers and Lee (1994), lime stabilization improves both the (apparent) cohesive (C_u) and frictional (Φ_u) components of the undrained shear strength as shown in Table 2.1. Lime will react with moderately fine and fine-grained soils to produce decreased plasticity, increased workability, reduced swell and increased soil strength. The clays provide pozzolanas, which are reactive substances responsible for the pozzolanic reaction. The pozzolanic reaction contributes to the long-term strength of the clay. In the construction industry, lime is traditionally used in mortars and plasters because of its superior plasticity, workability and other qualities. Lime's dominant use today is in soil stabilization where it upgrades low quality soils into usable materials. It saves cost of the project and reduces the use of granular materials.

Table 2.1: Strength of Lime-Clay Mixes (after Rogers and Lee, 1994)

Soil Type	Lime Content %	7 days	28 days	56 days
Undrained Shear Strength Cu, φu (kN/m^2, degrees)				
China Clay	0	-	45,6°	-
	1	230,27°	170,30°	270,37°
	3	120,36°	360,38°	410,46°
	5	475,40°	300,48°	1000,42°
Lower Lias Clay	0	-	41,7°	-
	4	720,62°	1860,48°	2450,50°
	6	600,61°	1800,48°	2300,54°
	8	1670,43°	3130,31°	5100,42°
Drained Strength C', φ' (kN/m^2 degrees)				
China Clay	0	-	7,16°	-
	1	125,48°	140,51°	200,50°
	3	125,52°	120,64°	240,60°
	5	275,60°	300,61°	310,63°
Lower Lias Clay	0	-	7,18°	-
	4	1025,43°	950,55°	1500,58°
	6	1175,56°	1225,57°	1400,62°
	8	1450,43°	1800,55°	3500,48°
Unconfined Compressive Strength (kN/m^2)				
China Clay	0	-	52	-
	1	317	269	526
	3	225	750	957
	5	1009	725	2245
Lower Lias Clay	0	-	83	-
	4	3132	4853	6601
	6	2330	5050	7449
	8	3803	5613	6554

2.2 History

The use of lime stabilization of clay in construction is over 5,000 years. The earlier stabilized earth roads were used in ancient Mesopotamia and Egypt, and that the Greeks and Romans used soil-lime mixtures. The first test involving soil stabilization was carried out in the USA in 1904. Lime was first used as a stabilizing agent of soil in modern construction practice in 1924 on short stretches of highway strengthened by the addition of hydrated lime. With the expansion of roads to cater for the growth of motor traffic in the 1930s, the use of stabilization of soils began to increase. It was extensively used during the Second World War for road and runway construction.

Today, stabilization of clay soil by incorporation of lime is a technique widely used throughout the world to improve its use in construction. Nowadays, it is used in road construction to improve sub-bases and sub-grades, for railroad and airport construction, for embankments, as soil replacement in unstable slopes, as backfill for bridge abutments and retaining walls, as canal linings, for improvement of soil beneath foundation slab and for lime piles (Bell, 1986). However, the use of lime stabilization method to stabilize the clay soil in Malaysia and around our region is not widely used. It need to have much more basic research with respect to Malaysian soils to establish the knowledge of lime stabilization method especially for road construction. Around this region, it may establish the improvement achieved from the lime stabilization method, however it does not seriously investigated the details specification in applying lime stabilization method for road construction. Also need to conduct the investigation of the performance of the lime-stabilized soil for a long-term period with the climatic condition of this region.

2.3 Lime for Soil Stabilization

Lime is an effective additive for clayey soils improving workability and strength. It provides an economic and powerful means of chemical improvement of heavy clays (Greaves, 1996). Various forms of lime have been successfully used for soil stabilization. Lime is a broad term, which is used to describe calcium oxide (CaO) – quicklime; calcium hydroxide $Ca(OH)_2$ – slaked or hydrated lime; and calcium carbonate ($CaCO_3$) – carbonate of lime. The relation between these three types of lime can be represented by the following equations (Sherwood, 1993):

$$CaCO_3 + \text{heat} = CaO + CO_2 \quad (2.1)$$

$$CaO + H_2O = Ca(OH)_2 + \text{heat} \quad (2.2)$$

$$Ca(OH)_2 + CO_2 = CaCO_3 + H_2O \quad (2.3)$$

The first reaction, which is reversible does not occur much below 5000°C and is the basis for the manufacture of quicklime from chalk or limestone. Quicklime may be high in calcium, magnesian or dolomitic, and of varying degrees purity. It may come in many sizes from very fine to very coarse. Hydrated lime is produced as a result of the reaction of quicklime with water (Equation 2.2). It is the hydrated form of lime, as a dry powder, putty or aqueous suspension (Little, 1995). Quicklime (by the reversal of Equation 2.1) and hydrated lime (Equation 2.3) will both revert to calcium carbonate on exposure to air. Only calcium oxide and hydrated lime react with soil. Calcium carbonate is not valuable for stabilization in civil engineering, although it is used in agriculture as a soil additive to adjust the pH of the soil.

In dolomitic limes, some of the calcium is substituted by magnesium. These types of lime can also be used for stabilization. Another types of lime are hydraulic limes, also known as grey limes, produced from impure forms of calcium carbonate, which also contain clay. They therefore contain less 'available lime' to initiate the effect on plasticity and strength. However, to compensate this, they contain reactive silicates and aluminates similar to those found in Portland cement. Thus, whilst their immediate effect may be less than that of high calcium limes in the long term, they may develop higher strengths.

2.4 Physical Properties of Hydrated Limes

The crystal structure of hydrated lime is a hexagonal-shaped plate or prism with perfect basal cleavage, but the physical particle is of varying size since the microscopic crystallites agglomerate in varying degrees (Little, 1995). There are four important physical properties of lime related to soil stabilization: specific gravity, bulk density, heat of solution and solubility.

2.4.1 Specific Gravity and Bulk Density

The apparent specific gravity is a more meaningful property because it represents the density of the actual material as it comes from the producer. Values of apparent specific gravity for quicklime vary from 1.6 to 2.8 with dolomitic quicklime averaging from 3 to 4 per cent higher. Average values of commercial oxides are 2.0 to 2.2. The range of specific gravities for different commercial hydrates is shown in Table 2.2 (Little, 1995).

Table 2.2: The range of specific gravity for different types of lime (after Little, 1995)

Types of lime	Specific Gravity
High calcium	2.3 – 2.4
Highly hydrated dolomitic	2.4 – 2.6
Normal hydrated dolomitic	2.7 – 2.9

The range of bulk density for quicklime is from 768 to 1120 kg/m^3 (48 to 70 pounds per cubic foot) with an estimated average of 880 to 960 kg/m^3 (55-60 pounds per cubic foot) for pebble-sized quicklime. Bulk density for commercially available hydrates ranges from 400 to 640 kg/m^3 (25 to 40 pounds per cubic foot) with an average value of approximately 564 kg/m^3 (35 pounds per cubic foot). The molecular weight of CaO is 56.08 and the molecular weight of $Ca(OH)_2$ is 74.10. Based on this ratio of molecular weights, it is apparent that in order to provide equal levels of CaO, it will require more $Ca(OH)_2$ than CaO (Little, 1995).

2.4.2 Particle Size

The typical sizes of quicklime vary from the maximum size of 203 mm (8 inches) in diameter (Lump Lime) to 25 mm (1 inch) sized pellets or briquettes. Hydrated lime is air classified to produce the fineness necessary. The normal grades of hydrated lime used for chemical purposes will have 75 to 95 percent passing the #200 sieve; while for special uses the hydrate may be classified as fine as 95 percent passing a #325 sieve. Due to the air classification, generally the commercial hydrated lime produced is purer than the quicklime from which it is derived since much of the impurities are rejected in the classifier (Little, 1995).

2.4.3 Heat of Formation

The heat of formation is synonymous with the heats of hydration and reaction. For commercial hydrates, the heat of hydration for $Ca(OH)_2$ is approximately 6.396×10^7 J/(kg.k) (27,500 Btu/lb mole) or 1.14×10^6 J/(kg.k) (488 Btu/lb mole) for quicklime. For $Mg(OH)_2$, the values are somewhat lower, 3.35×10^7 to 4.2×10^7 J/(kg.k) (14,400 to 18,00 Btu/lb mole). This substantial heat of hydration is most important in the production of hydrated lime or quicklime slurries (Little, 1995).

2.4.4 Solubility of Hydrated Lime

The solubility of $Ca(OH)_2$ is 1.330 g CaO/l of saturated solution at 10°C in distilled water. At 0°C, the solubility will increase to 1.4 g CaO/l. $Ca(OH)_2$ is approximately 100 times more soluble than calcium carbonate. The solubility of lime expressed as CaO or $Ca(OH)_2$ at different temperatures in g/100g of saturated solution as presented in Table 2.3.

The solubility of $Ca(OH)_2$ is affected by some salts and inorganic chemical solutions in varying degrees. Most of the salts will increase the solubility of hydrated lime by about 10 to 15 percent. Generally, the increasing of temperature still depresses solubility of the hydrate (Little, 1995).

Table 2.3: Solubility of lime at different temperatures expressed in g/100g of the saturated solution (after Little, 1995)

Temperature, °C	*Solubility of CaO, g/100g*	*Solubility of $Ca(OH)_2$, g/100g*
0	0.140	0.185
10	0.133	0.176
20	0.125	0.165
30	0.116	0.153
40	0.106	0.140
50	0.097	0.128
60	0.088	0.116
70	0.079	0.104
80	0.070	0.092
90	0.061	0.081
100	0.054	0.071

2.5 Chemical Properties of Hydrated Limes

Hydrated lime is more stable than quicklime since water does not cause a change in its composition. The primary factor influencing the stability of hydrated lime is carbon dioxide which reacts with hydrated lime to form calcium carbonate (Little, 1995), generally in slow rate.

2.5.1 pH of Lime-Water Solutions

The pH of solutions at 25°C (77°F) rise sharply with the addition of very low concentrations of $Ca(OH)_2$. A concentration of only approximately 0.064 g/l of hydrated lime will increase the pH of distilled water from 7 (neutrality) to above 11. The pH of the solution peaks at approximately 12.45 at 25°C (77°F). The rise of temperature will reduce the solubility of $Ca(OH)_2$ and therefore, decrease the pH slightly. The high pH of a lime-water solution is of great practical importance in soil stabilization. This is because a high pH or basic environment increases the ability of the lime to react with soil minerals and produce cementitious product, which can stabilize particles by 'gluing' them together (Little, 1995).

2.5.2 Rate of Solution

Lime in solution produces a high pH environment, which can react with soil minerals. There are many factors for the rate and efficiency of the reaction between lime and water. The most important factors in soil stabilization are the particle size of the lime and the nature of the solute. The rate of solution of hydrated lime increases with higher specific surface areas of hydroxide particles. If there are organic solutes, such as sugar, acids and salt, it will have a profound effect on lime solubility and the rate of solution (Little, 1995).

2.5.3 Reaction of Lime With Carbon Dioxide

When the carbon dioxide (CO_2) reacts with lime, either quicklime (CaO) or hydrated lime ($Ca(OH)_2$), it will result in the formation of calcium carbonate ($CaCO_3$). This process should be minimized during construction as it disturbs the system of lime in the CaO or $Ca(OH)_2$ form, which is reactive with the soil minerals (Little, 1995). This is undesirable as it reduces the available lime content. Although the rate of carbonation is very slow with full carbonation likely to occur only after several days of continuous exposure to an unlimited flow of air, the lime should be protected in transport and storage. Lime treated soils should be sealed by rolling as soon as possible after mixing to minimize atmospheric carbonation (Lime Stabilization Manual, 1990).

2.5.4 Reaction With Silica and Alumina

Lime reacts with many compounds and elements including sulfur compounds, acid gases, halogens, magnesia compounds, iron, phosphorous, silica and alumina and metals. The reactions between lime and available silica and alumina are quite complex and it has been well documented that CaO reacts with silica (SiO_2) and alumina (Al_2O_3) and water at elevated temperatures (93°C to 260°C, 200°F to 500°F) to form hydrated calcium silicate and calcium-aluminate compounds (Little, 1995). The cementing agent is thus exactly the same as for ordinary Portland cement, the difference being that with the latter the calcium silicate gel is formed from hydration of anhydrous calcium silicate (cement), whereas with the lime the gel is formed only after attack on and removal of silica from the clay minerals of the soil. The concept with cement stabilization is that the latter is essentially independent of soil type: as illustrated by Figure 2.1 which shows the rate of gain of strength for cement stabilized soils to be constant where as the rate for lime stabilized soils is different for each soil type (Ingles and Metcalf, 1972).

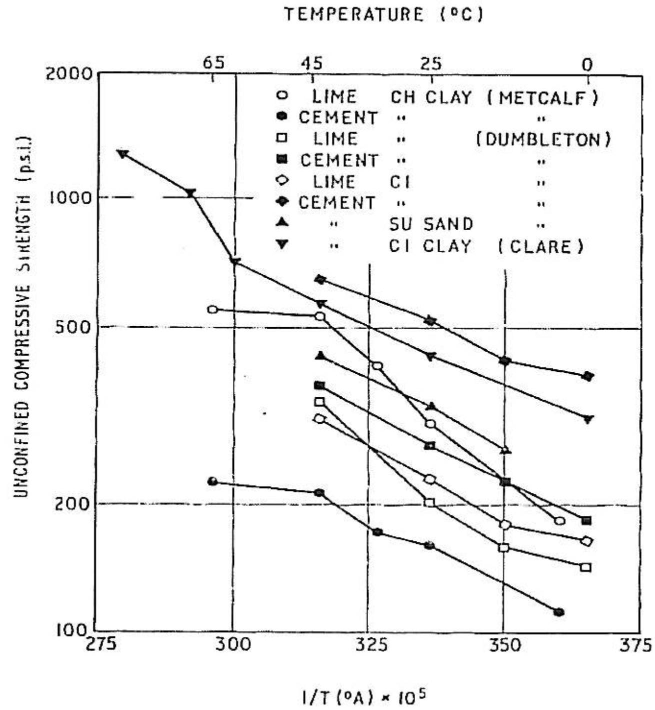


Figure 2.1: **The relation between rate of gain of strength and temperature for lime and cement stabilized soils (after Ingles and Metcalf, 1972)**

2.5.5 Environmental Effects

Lime is not toxic to workers in construction, manufacturing or lime consuming plants nor is its air-borne dust particles harmful to the public. Lime's greatest use is for environmental cleanup of water, wastewater, air and solid wastes. When lime comes into contact with water, a chemical reaction takes place (hydration). This reaction releases considerable heat. The resultant hydrated lime is a caustic alkali in the presence of water and can cause chemical burn. From the high alkalinity of lime products, safety procedures and equipments are encouraged as outlined in the Lime Stabilization Manual (1990).

2.6 Lime Industry

There are many uses of lime in the industry as shown in Figure 2.2. The dominant role of lime is in steel production. Besides that, it is also used in non-ferrous metallurgy, chemical industry, paper industry, refectories, sugar refining, agricultural liming, glass making, leather tanning, plastic and pigment production, environmental uses for potable water softening and clarification. In construction, lime's traditional use in mortar and plaster still flourishes. However, lime's largest construction use in the stabilization of roads, airfields, building foundations, earth dams, etc., where it upgrades low quality clayey soils into satisfactory base and sub-base materials (Little, 1995).

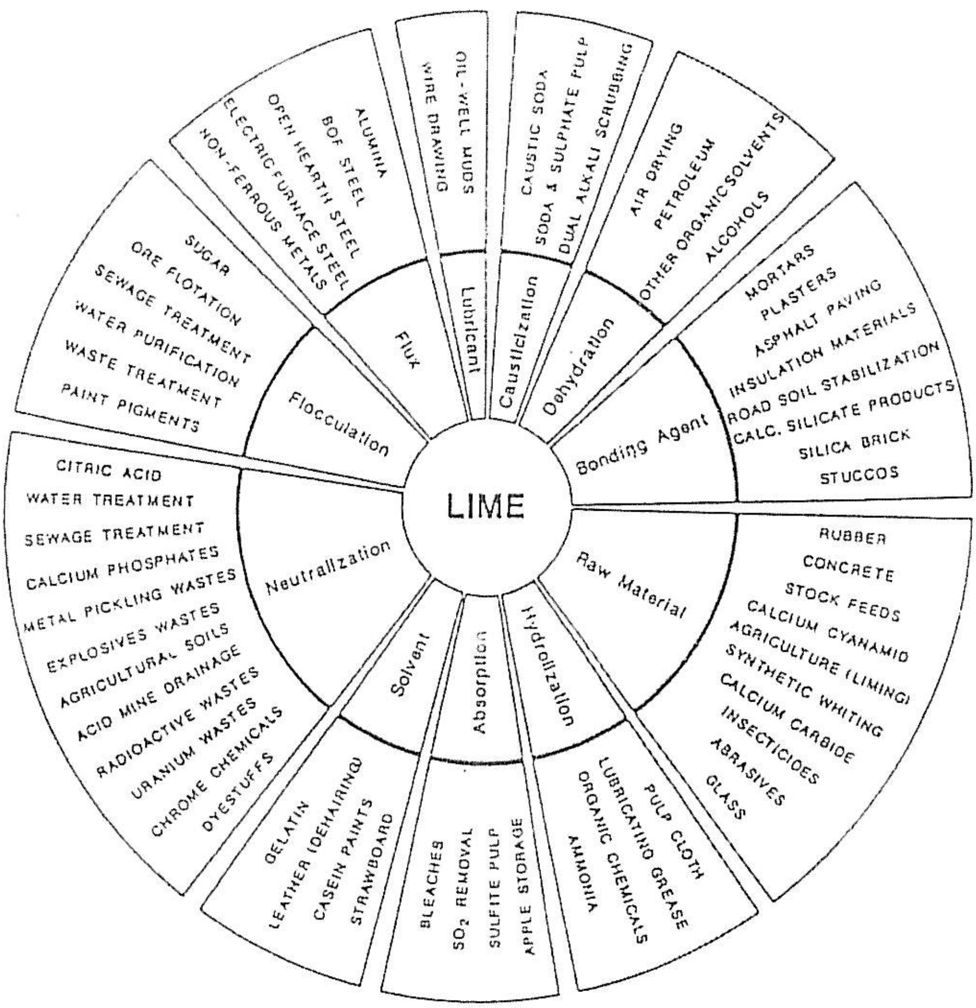


Figure 2.2: The use of lime in industry (after Little, 1995)

2.7 Lime-Clay Reaction

As the oxide reacts immediately with any available water to form hydroxides, the main reactions between all the common lime types and soils are alike. It is recognized that lime has an immediate effect on clay soils, improving its granulation and handling properties and long term effect on strength, causing continuing strength improvement with time (National Association of Australia State Road Authorities, 1986).

2.7.1 Mechanism of Lime-Soil Reaction

The addition of lime to a fine-grained soil initiates several reactions. The initial reaction is the drying out process by absorption and evaporation. This reaction will reduce the moisture content of the wet soil. After that, the rapid physico-chemical reactions between the lime and clay minerals produce immediate changes in soil plasticity. The value of this reaction can be appreciated by consideration of the soil in Figure 2.3 at a moisture content of 35 per cent. Since the plastic limit of the soil is 25 per cent, it is clear that at moisture content of 35 per cent it will be in a wet and sticky condition, impossible to compact and impassable to traffic. After the addition 2 per cent of lime, it will change the plastic limit to 40 per cent so that the moisture content of the soil will be 5 per cent below, (y in Figure 2.3), instead of being 10 per cent above the plastic limit (x in Figure 2.3). Lime reacts with the clay minerals of the soil, or with any other fine pozzolanic component such as hydrous silica, to form a tough water-insoluble gel of calcium silicate, which cements the soil particles (Ingles and Metcalf, 1972). The silicate gel proceeds immediately to coat and to bind clay lumps in the soil and it will block off the soil pores in the manner shown by Figure 2.4.

2.8 Previous Study on lime-clay Response (Lime Stabilization)

When sufficient lime is added to a soil in the presence of water, the pH value of the solution is raised to about 12.4. The high alkaline environment promotes the dissolution of silica and alumina from the clay particles. The calcium ions from the lime then will react with alumina and silica to form soluble gels of Calcium Silicate Hydrate (CSH) and Calcium Aluminate Hydrate (CAH) (Holt and Freer-Hewish, 1996). These cementitious products will cement the soil particles and proceed to coat and bind the clay lumps in the soils and block off the soil pores. The gel then gradually crystallizes to form an interlocking structure (Al-Mhaidib and Al-Shamarani, 1996).

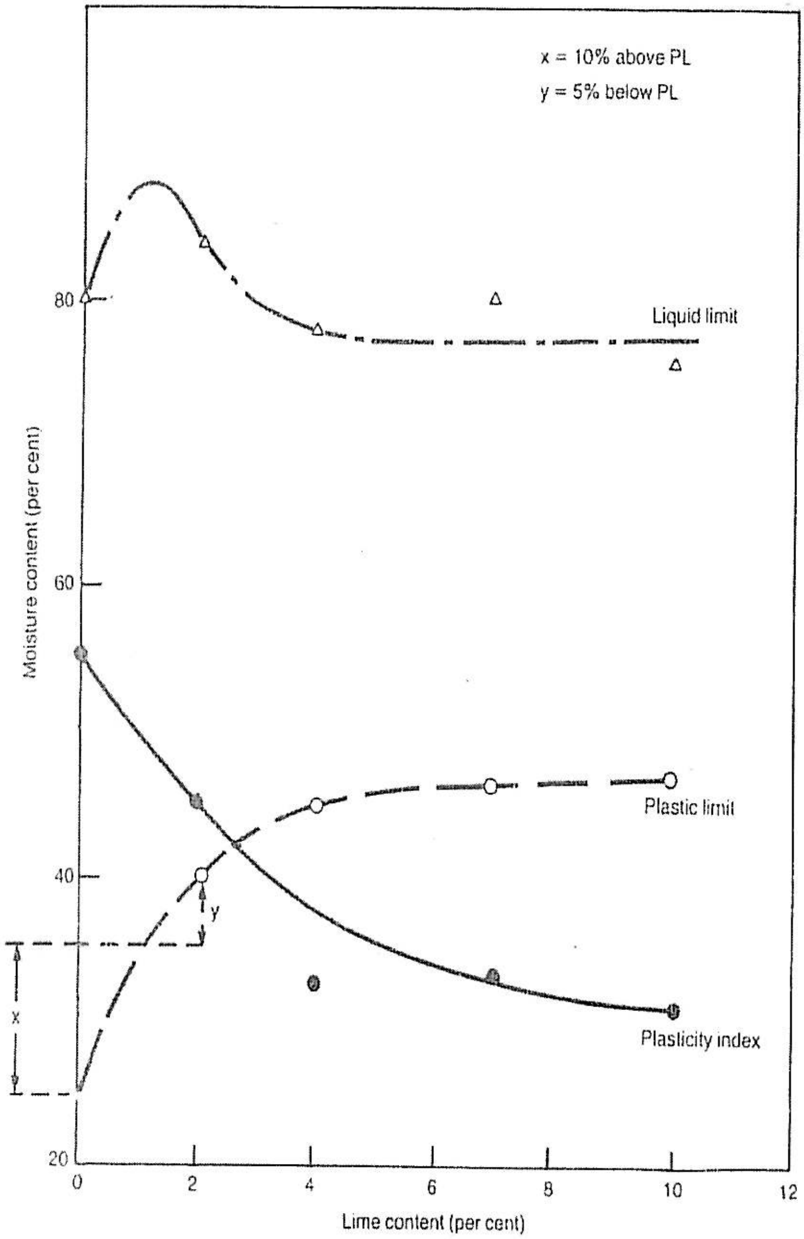


Figure 2.3: Effect of the addition of lime on the plasticity of London clay (after Sherwood, 1993)

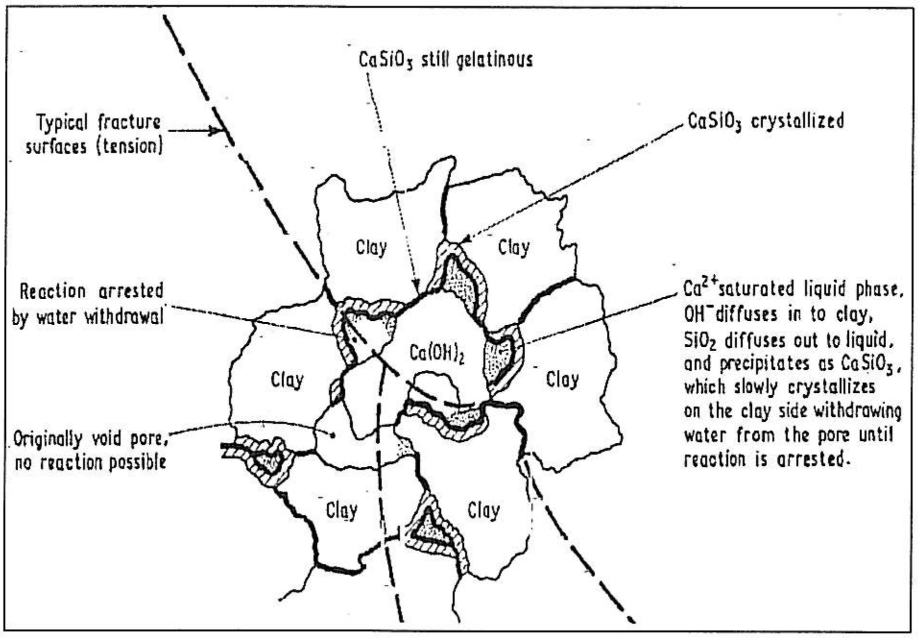


Figure 2.4: Mechanism of lime stabilization of clay soils (after Ingles and Metcalf, 1972)

This pozzolanic reaction only occurs with the presence of water, which is able to carry calcium and hydroxyl ions to the surface of the clay particles. In addition, the pozzolanic reaction is apparently effective at the age of 14 days of curing period, and may require a year to complete. It depends on mineralogy and the reaction conditions, including temperature, moisture and curing conditions.

An enhanced stabilizing effect may be obtained by leaving the material loose or by breaking up lightly compacted material and then recompacting the material after a 24 hour delay. This is advantagers in terms of field work as urgency of lime compaction is not critical during construction. Lime-soil mixtures do not need rapid cementation akin to the setting of concrete because the effect of delay in compaction is far less important with lime stabilization as shown in Figure 2.5 (Ingles, 1972).

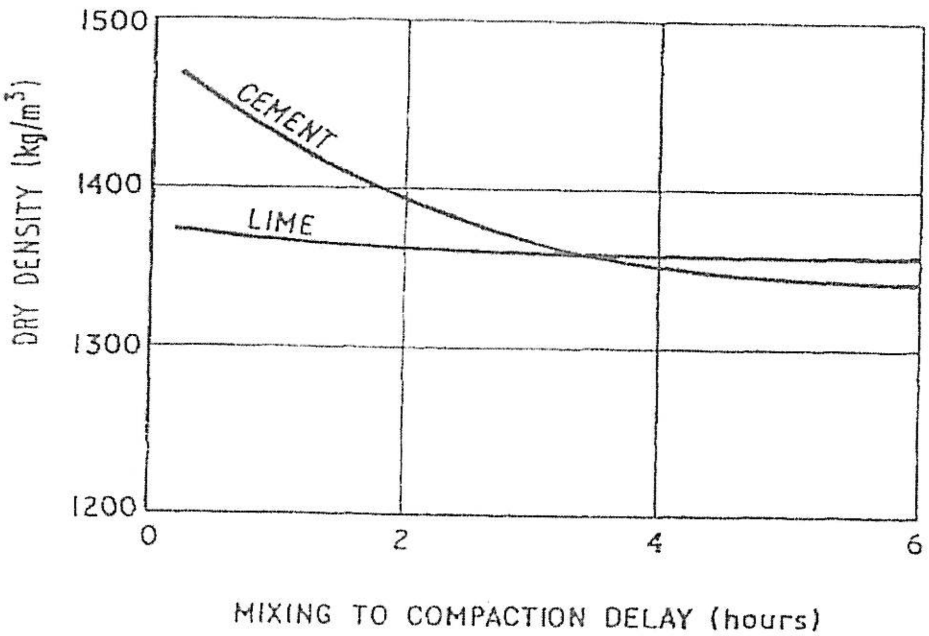


Figure 2.5: Effect of delay on the compacted density of a heavy clay, stabilizer content 10 per cent (after Ingles, 1972)

2.8.1 Continuous Increase in Soil Strength

The cured soil-lime mixture exhibits a higher CBR value and enhanced compressive and shear strengths characteristics. The unconfined compressive strength of soil-lime mixtures increases with increasing lime content to a certain level, usually about 8 per cent for clay soil. The rate of increase then diminishes until no further strength gain occurs with increasing lime content as shown in Figure 2.6. The gain in strength with time of a compact soil-lime mixture broadly follows the pattern shown in Figure 2.7 (Bell, 1988).

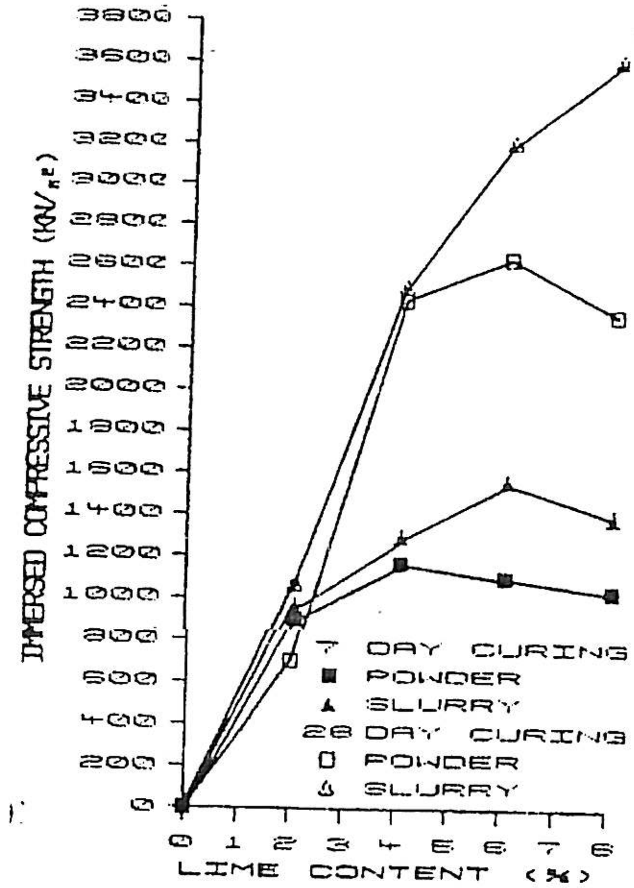


Figure 2.6: Effect of the addition of lime, in slurry and powder form, on the strength of silty clay (after Bell, 1988)

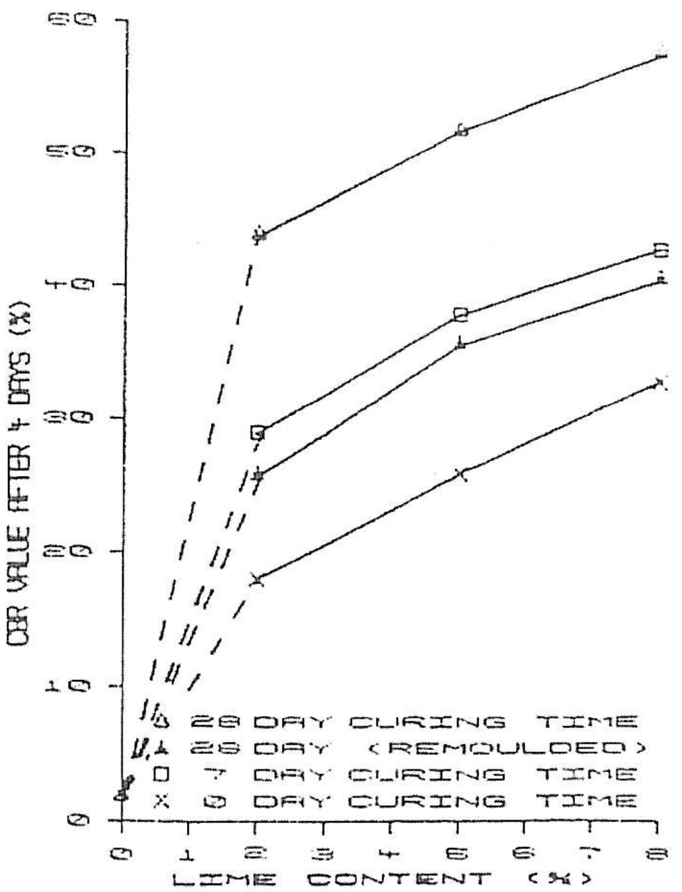


Figure 2.7: **Effect of lime content on strength for various soils stabilized with hydrated lime, cured for 7 days at 25°C having constant moisture content (after Bell, 1988)**

2.8.2 Less Deflection Under Load

The significant result after treatment with lime in the deflection behaviour is shown in Figure 2.8. It shows that the treated soil has been enhanced in term of stiffness after the stabilization process.

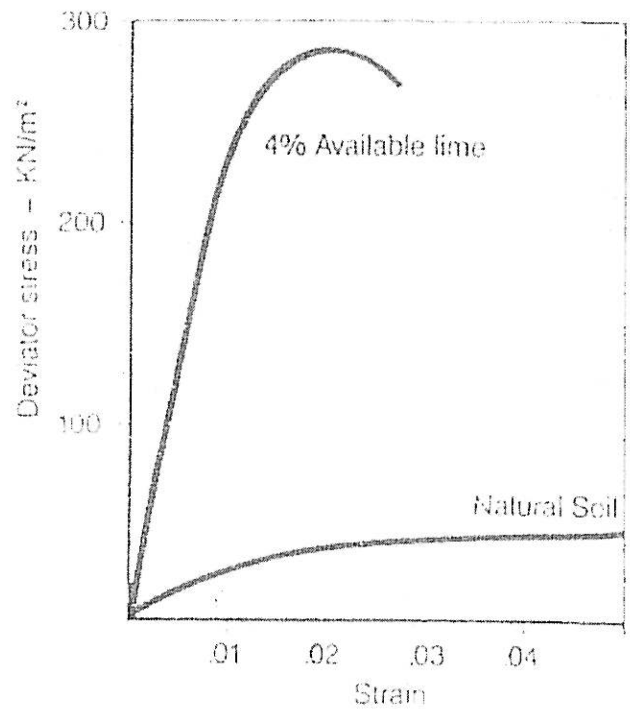


Figure 2.8: Typical stress-strain curve showing effect of lime stabilization (after Lime Stabilisation Manual, 1990)

2.8.3 The Treated Soil is Less Susceptible to Moisture Condition

As illustrated in Figure 2.9, the addition of 3 % available lime halves the shrinkage experienced after 28 days of curing. It is due to the reduction of volume changes caused by fluctuations in moisture content for lime treated clay (Lime Stabilisation Manual, 1990).

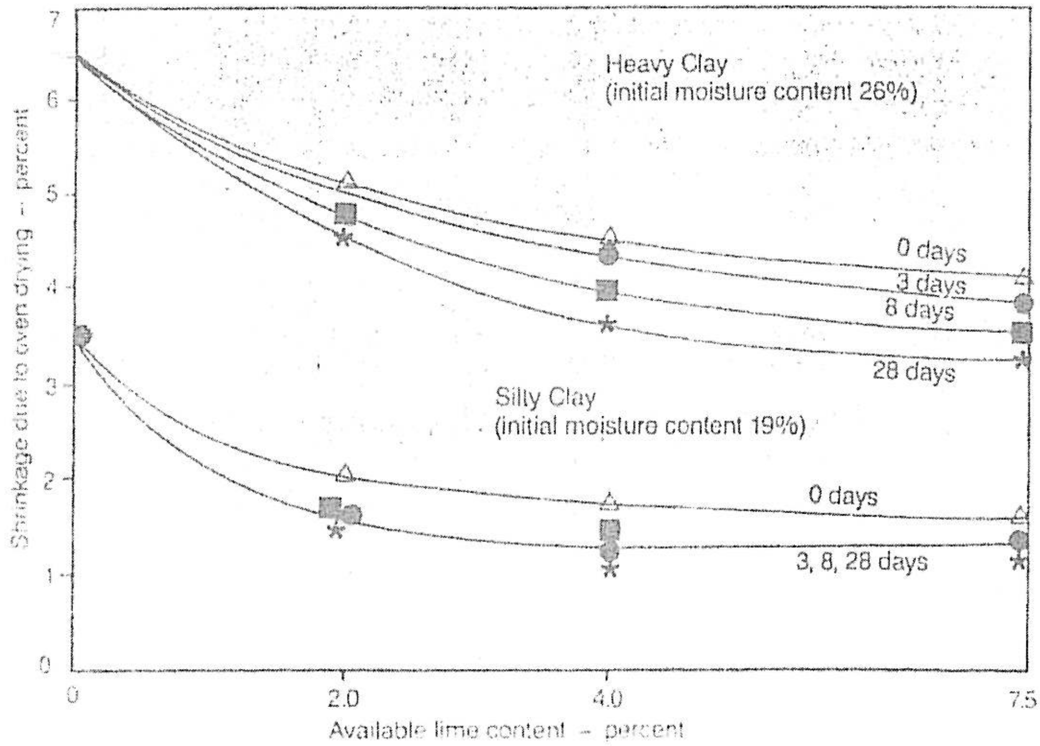


Figure 2.9: **Effect of available lime content on the linear shrinkage of two stabilized soils on drying at 105°C (after Lime Stabilisation Manual, 1990)**

2.8.4 Re-instatement

According to Lime Stabilisation Manual (1990), soil-lime mixture can exhibit self-healing properties. This ability to regain strength for any distress caused by excessive loading would be cumulative. Hence, it is usual to re-shape and compact during favorable conditions to restore the stability of the material. Likewise, trenches through lime stabilized clay soils can be re-instated with the excavated material, which will handle more like a granular soil and benefit from subsequent strength gain.

2.9 Role of Sub-Grade In The Pavement System

The natural soil on which the pavement is constructed is termed the sub-grade and generally its strength is of prime importance to the design of the layer above the formation. Most sub-grades, especially those consisting of clays and silt become weaker as their moisture content increases and therefore an important function of a pavement is to provide a 'roof' to the sub-grade (Lilley, 1991).

A pavement surface course is generally one of two forms: Portland Cement Concrete (PCC) or asphalt concrete (AC). The purpose of both courses is to provide a smooth and safe riding surface. PCC often referred to as a rigid pavement requires uniform support to function properly and economically. After the long-term load transfer from the vehicle and the cracks developed in response to shrinkage or temperature, PCC pavement requires uniform and stable support of the sub-base directly below the PCC slab as shown in Figure 2.10. Lime stabilization method can be used to help provide this type of support (Little, 1995).

Base on Figure 2.10, the sub-base must have enough strength and enough resistance to counter the pumping effect from the top. The action of subsequent pumping may lead to the loss of material directly below joint or crack and lead to loss of support which, in turn, leads to further cracking and joint deterioration. If the sub-base is supported by a weak sub-grade, settlement will occur and the sub-base will deform. Therefore, the sub-base should be laid on a strong and stiff layer of sub-grade to have enough compaction during construction and good support. The lime-stabilized sub-grade as a capping layer can provide the uniform support and resist permanent deformation before the construction of the sub-base.

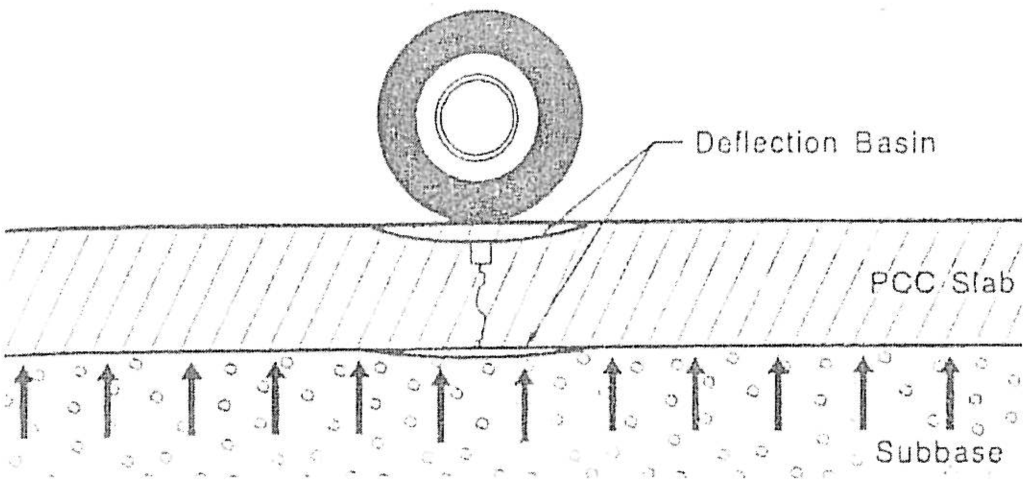


Figure 2.10: Sub-base below PCC pavement provides uniform support and aids in load transfer (after Little, 1995)

Asphalt concrete is also referred to as a flexible pavement. The base and sub-base are major structural components of a flexible pavement (Little, 1995). Figure 2.11 illustrates the critical stresses in a typical flexible pavement with the hot mix asphalt concrete (HMAC) surface. The HMAC is supported by an unbound aggregate base course (ABC), which is supported by a lime-stabilized sub-grade (LSS).

The flexural tensile stresses and strains induced in the asphalt concrete layer by traffic loads are related to fatigue cracking. If these stresses and strains are too large, the result is a pavement with asphalt cracking in the wheel path. The shearing stresses in the HMAC are related to distortion, shoving or rutting within the surface layer. These stresses are induced by the vertical contact stresses of the wheel load as well as the rolling and braking surface shearing stresses. The vertical compressive stresses and strains within the granular bases, sub-bases and sub-grades are related to rutting and pavement roughness developed in the base and sub-base layers.

The magnitude of the stresses and strains developed in the pavement layer is dependent upon the magnitude of the contact stresses of the wheel load, the thickness of the respective layers and the relative stiffness or moduli of the various layers. Since the magnitude of the stresses developed within the asphalt layer is heavily influenced by the modulus ratio (or stiffness ratio) between the surface and the base course, it is practically to strengthening the base or sub-base to minimize distress within the HMAC. The increased stiffness of the base will provide better support for the surface. This situation will reduce the tensile flexural stresses and the shearing stresses developed within the HMAC and overcome the potential of fatigue crack or deformation (Figure 2.11).

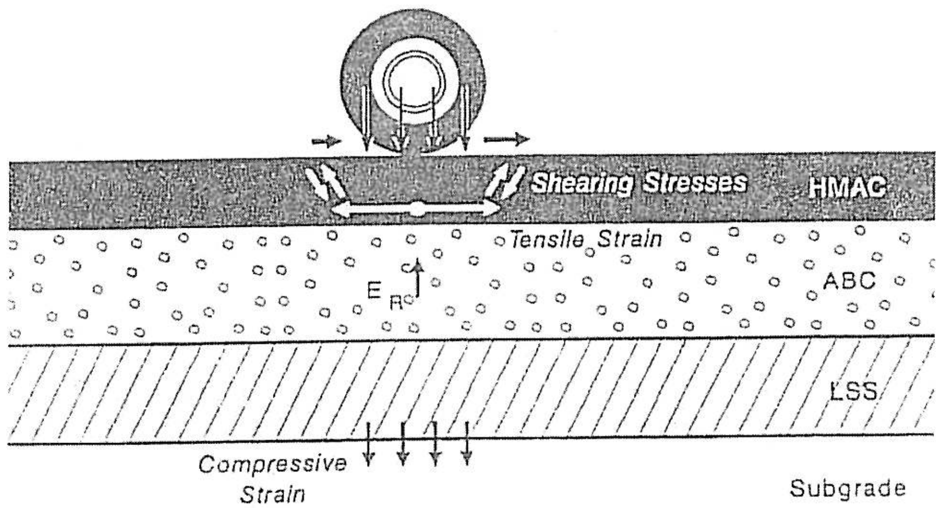


Figure 2.11: Critical parameters in a flexible pavement include flexural tensile strain in the HMAC, shearing stresses and strains at the top of the sub-grade (after Little, 1995)

In general, when the stress or confinement within the layer is increased, the response modulus or stiffness increases. Development of a high confining stress within the granular base or sub-base can be achieved by improved support of sub-grade. Lime stabilization method improves the consistency of the soil over a wide range of moisture contents; it reduces plasticity, reduces swell potential and volume change potential and increases strength of the soil. This means that a lime-stabilized sub-grade can provide improved support and more consistent support for the sub-base or base compared to the unstabilized, natural soil (Little, 1995).

2.10 Role of Sub-grade in Minimizing Pavement Distress in Flexible Pavements

The magnitude of the stress and strain developed within the various pavement layer is dependent upon the magnitude of the contact stress of the wheel load, the thicknesses of the respective layers and the relative stiffness or moduli of the various layers. It shows that the magnitude of the stresses developed within the asphalt layer is heavily influenced by the modulus ratio (or stiffness ratio) between the surface and the base course. The stress within Hot Mix Asphaltic Concrete (HMAC) can be minimized by strengthening the base or sub-base. In other words, the increased stiffness of the base or sub-base provide better support for the surface and as a result, both tensile flexural stresses and shearing stresses developed within the HMAC are reduce as shown in Figure 2.12.

Therefore, with improvement from lime stabilization method, lime-stabilized sub-grade can give good support and more consistent compare to the unstabilized soil. This will reduce the cracking and shearing distress within the HMAC (Little, 1995).

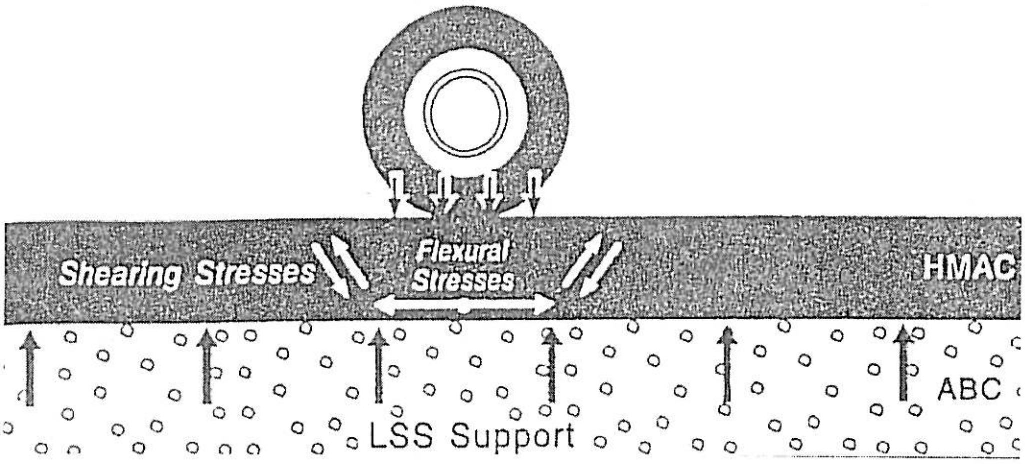


Figure 2.12: Tensile strains and stresses in the Hot Mix Asphaltic Concrete (HMAC) are reduced as a result of improved support of the HMAC and aggregate base course (ABC) by the lime stabilized sub-grade (LSS) layer (after Little, 1995)

2.11 The Use of Stabilized Materials in Road Pavement Layers

A number of structural criteria must be satisfied to give satisfactory service of a pavement. Firstly, the sub-grade must be able to sustain traffic loading without excessive deformation; this is controlled by the vertical compressive stress or strain at formation level. Secondly, the materials used in road-base design for long life must not crack under the influence of traffic; this is controlled by the horizontal tensile stress or strain at the bottom of the road-base. Then, the deformation of the bituminous materials of the pavement must be limited. Their deformation is a function of their creep characteristics. Finally, the load spreading ability of the sub-grade and capping layers must be adequate to provide a satisfactory working platform (Sherwood, 1993). The use of lime stabilization for sub-grade improvement and for the construction of the capping in road construction can lead to economies. It may be the solution when no other suitable materials are readily available.

2.11.1 Capping Layers

On weak sub-grades it is common practice to use a capping between the sub-grade and the sub-base. This will reduce the thickness of the sub-base which would otherwise be required and provides a suitably firm surface for the placement and compaction of the sub-base. The capping layer can be constructed from a low-cost granular material to a lower specification than the sub-base itself. As an alternative to a granular material, the in situ stabilization of the sub-grade with lime may offer economic and environmental advantages (Sherwood, 1993).

In addition to the weakening of many types of sub-grades caused by weathering, the passage of construction traffic can also be harmful. This problem is of greatest significance on sub-grades which are naturally weak, such as silts and clays. If these soils are disturbed, their bearing value is usually reduced and is rarely recovered subsequently. It is now British practice during the construction of major roads either to cover such soils with a low-cost, unbound granular material or to stabilize them to provide a 'working platform' and protect the natural sub-grade from disturbance and weathering. This 'capping layer' also simplifies the design and construction of overlying strata by creating a layer with a uniform bearing value throughout the site (Wignal, 1991).

Table 2.4: Capping layer and sub-base thickness (after Wignal, 1991)

Ground support		Sub-base thickness (mm)	Capping layer thickness (mm)
Good	CBR 15–30%	150	–
	CBR 5–15%	225	–
Normal	CBR 2–5%	150	350
Poor	CBR $\leqslant$ 2%	150	600

DEFINITIONS

Good support: Granular material not particularly susceptible to moisture and capable of supporting construction plant used for laying and compacting the sub-base.

Normal support: Moisture-susceptible material such as silts and marls and over-consolidated clays in cuttings and embankments.

Poor support: Very moisture-susceptible materials such as silty clays in cuttings and embankments, silts and marls at grade or in cutting, and other susceptible formations when the water table can be within 300 mm of the formation level.

The functions of a capping layer is to protect the sub-grade from the adverse effect of wet weather. It also provides a working platform on which sub-base construction can proceed with minimum interruption in wet weather. The capping layer will allow the full load-spreading capabilities of the sub-base, which would not be possible if the sub-base is laid directly on the top of the weak sub-grade. When the CBR of the sub-grade is less than 15 %, British design criteria require the use of a suitable capping layer of a low –cost local material. The recommended thicknesses of capping, which should have a minimum CBR value of 15 % are given in Figure 2.13 and 2.14 (Sherwood, 1993). Table 2.4 also give the recommendation of the capping layer and sub-base thickness with the condition of the formation support.

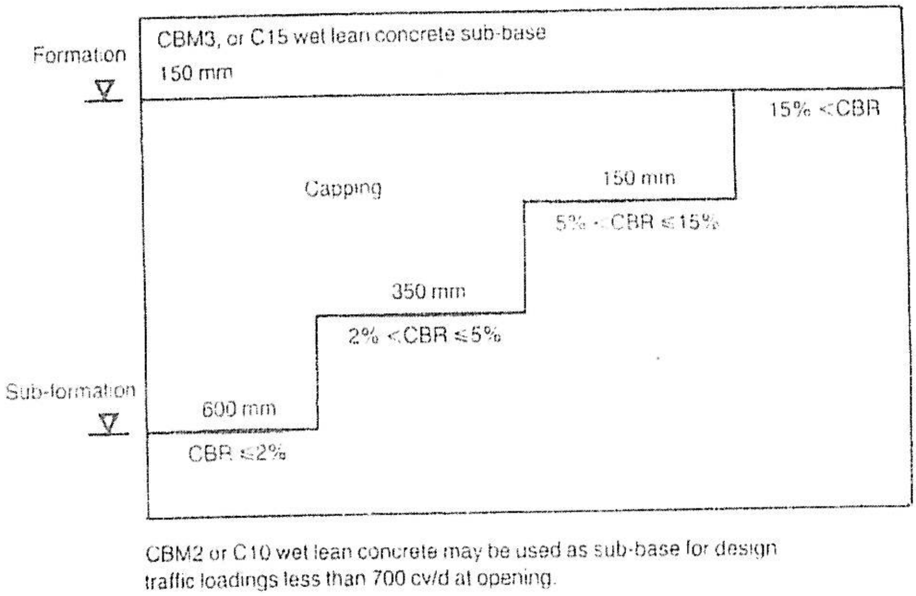
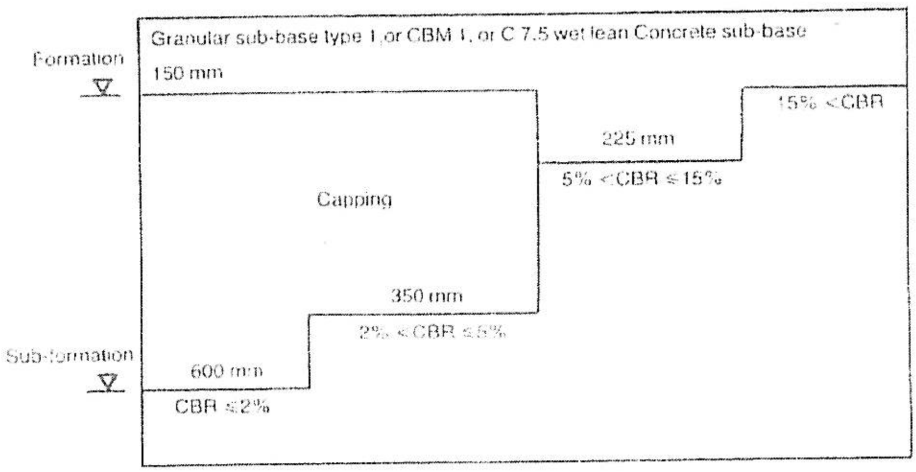


Figure 2.13: Design thickness for flexible pavement and composite pavement of capping and sub-base for different CBR values of sub-grade (after Sherwood, 1993)



Granular sub-base Type 2 may be usec as sub-base for design traffic loadings less than 400 cv/d at opening
Granular sub-base Type 2 shall have a CBR of 30% or more when tested in accordance with clause 804

Figure 2.14: Design thickness for rigid and concrete pavement of capping and sub-base for different CBR values of sub-grade (after Sherwood, 1993)

From Figure 2.13, it is shows that a minimum thickness of 150 mm and a maximum thickness of 600 mm of the capping layer is required for road construction. This thickness is dependent on the CBR value of the soil.

2.11.2 Base and Sub-base

The base and sub-base are major structural components of a flexible pavement. In order to perform acceptably, bases and sub-bases must provide adequate support for pavement surface and load-distributing power expected to protect the underlying sub-grade layer from being overstressed. Therefore, strengthening the base and sub-base layers through lime stabilization provides improved load-spreading capability of the pavement structure and hence protects the sub-grade from being overstressed by traffic loading. Then, the most valuable result here is that the potential to develop pavement roughness or deep layer rutting is reduced (Little, 1995).

2.12 Ultimate Bearing Capacity for Strong Soil Underlain by Weaker Soil

The concept of road design is based on the ultimate bearing capacity of the lime stabilized capping and soft clay structure. In road construction, layered soil profiles are often encounterd. In such instance, the failure surface at ultimate load may extend through two or more soil layers. In this thesis, two layers of soil structures were investigated which is the lime stabilized soil and soft clay layer. The ultimate bearing capacity was measured from the model testing and compared with the value based on theory. Braja (1999) suggested the ultimate bearing capacity (q_u) for the foundation in stronger saturated clay as a top layer and weaker saturated clay as a bottom layer can be calculated using the Equations 2.4 and 2.5.

$$q_u = \left(1 + 0.2\frac{B}{L}\right)5.14C_2 + \left(1 + \frac{B}{L}\right)\left(\frac{2C_a H}{B}\right) + \gamma_1 D_f \leq q_t \tag{2.4}$$

$$q_t = \left(1 + 0.2\frac{B}{L}\right)5.14C_1 + \gamma_1 D_f \tag{2.5}$$

where,

q_u	=	Ultimate bearing capacity
q_t	=	Bearing capacity of the stiff layer
B	=	Width of the foundation
L	=	Length of the foundation
C_1	=	Undrained shear strength of the stiff layer
C_2	=	Undrained shear stength of the soft layer
C_a	=	Adhesive force
H	=	Depth of stiff layer
γ_1	=	Unit weight of stiff layer

D_f = Depth of foundation in the stiff layer

2.13 Road Design Concept

The primary function of the pavement structure is to reduce and distribute the surface stresses (contact tyre pressure) to an acceptable level at the sub-grade. A flexible pavement reduces the stress by distributing the traffic wheel loads over greater and greater areas, through the individual layers until the stress at the sub-grade is at an acceptable low level. The traffic loads are transmitted to the sub-grade by aggregate-to-aggregate particle contact. Confining pressures (lateral forces due to the material weight) in the sub-base and base layers increase the bearing strength of these materials. A cone-distributed load reduces and spreads the stresses to the sub-grade, as shown in Figure 2.15. For simplicity, the pyramidal spread of the load can be considered to be at 45° to the horizontal and this gives an approximately correct stressing figure. In reality, the spread is slightly greater in the upper layers of the road structure.

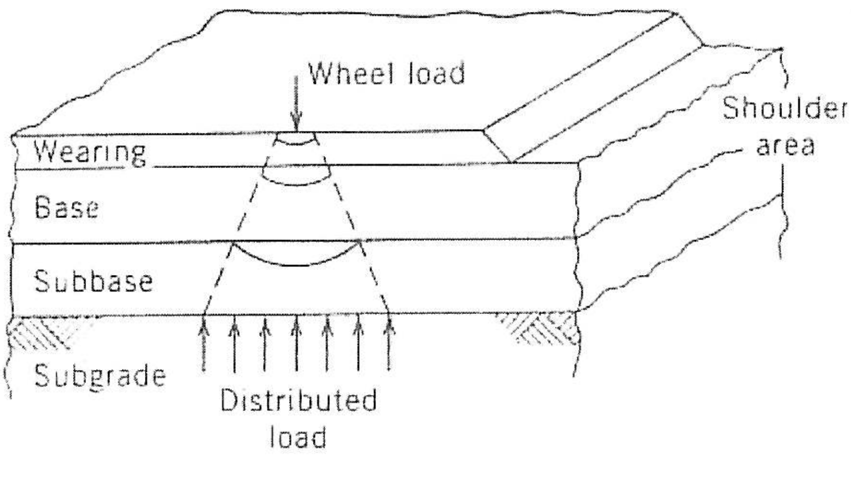


Figure 2.15: Distribution of load on a flexible pavement (after Mannering and Kilareski, 1998)

The structural performance of a pavement is aimed at distributing the load under the wheels of vehicles over larger areas to prevent stressing, beyond its load-bearing capacity, the native soil (or sub-grade) on which the pavement system is constructed (Papacostas and Prevedouros, 2001). As shown in Figure 2.16, the degree of load distribution decreases from the top to the bottom of the pavement structure.

The main purpose of the road structure is to provide a means of reducing the stress or pressure due to a wheel load to a value, which the ground under that structure can support. The interest in the static and dynamic stresses is greatest at the surface of the road. The stresses spread in a pyramidal shape throughout the depth of the structure. As the spread of the load increases, the stresses reduce, until at the formation level, the stresses are low enough for the natural ground to support it without distortion or damage (Wignall *et.al*, 1991).

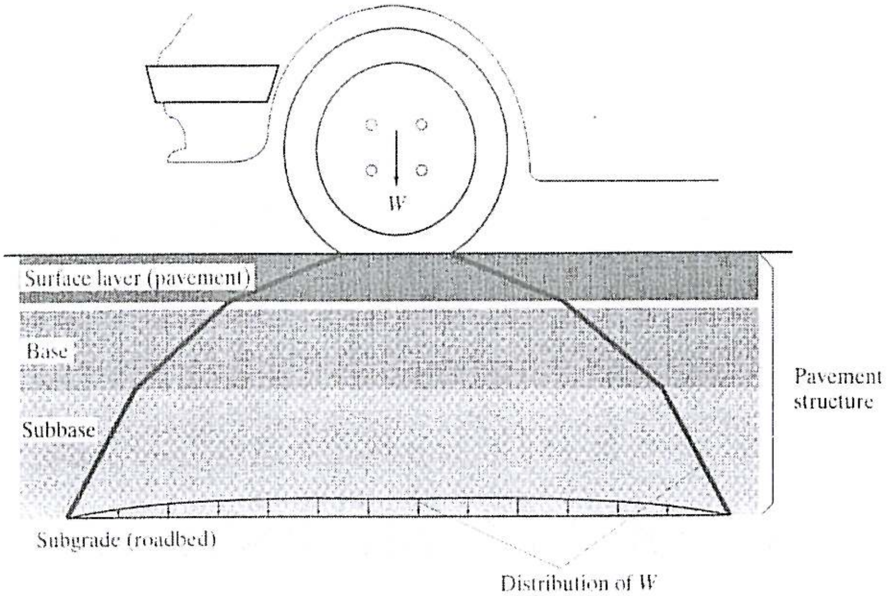


Figure 2.16: Distribution of weight of wheel from the contact area to the native soil (after Papacostas and Prevedouros, 2001)

Pavement design has been aptly defined as the process of developing the most economical combination of pavement layers (in relation to both thickness and type of materials) to suit the soil foundation and the cumulative traffic to be carried during the design life (Rao, 1996). In a flexible pavement, the pavement structure is expected to deform in the same way as the sub-grade through lateral distribution of the applied load with depth. Thus, the basic requirement in the design of these pavements is interposing layers of sufficient thickness to distribute the wheel load to the sub-grade, without causing any overstressing in the sub-grade. As such, strength of the sub-grade plays an important role in design (Figure 2.17).

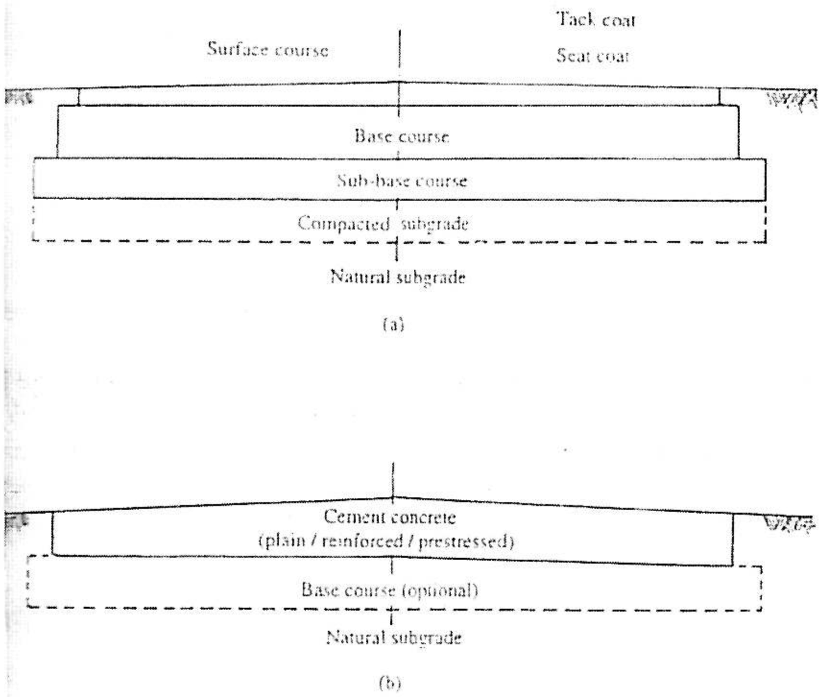


Figure 2.17: Components of pavement structure (a) Flexible pavements (b) Rigid pavements (after Rao, 1996)

From Figure 2.18, it shows that the basic approach for pavement design was based on the strength of the sub-grade. The pavement thickness can be reduced when the strength of the sub-grade increased.

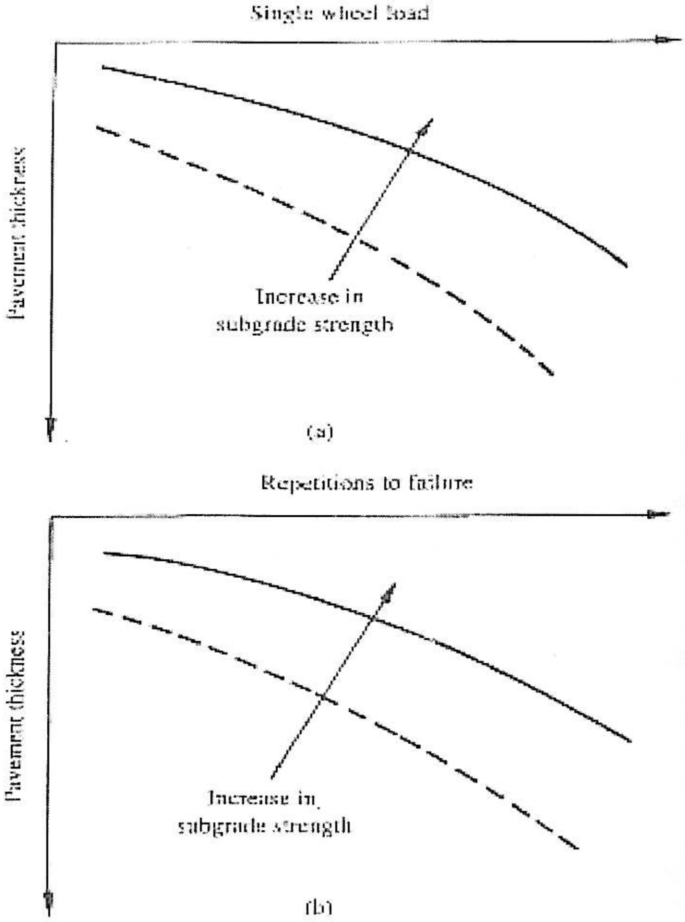


Figure 2.18: Basic approaches for pavement design (after Rao, 1996)

Despite the advances in highway engineering, a rational method of design of pavement has yet to be evolved. Most methods are either empirical (based upon past experience) or based on full-scale observations. Methods based on theory are still in the developmental stages. In developing a rational design procedure, knowledge of stress distribution in multilayered media is essential.

In the theories of stress distribution, the assumptions for each layer that have been considered are:

a) Homogeneous, isotropic and elastic (with a characteristic E and μ)

b) Has a definite thickness except the lowest layer which has semi-infinite thickness.

c) Has full friction developed at interfaces and

d) Has no surface shearing force.

Rao (1996) mentioned that Boussinesq first published the stress distribution theory in 1885 for the case of a point load, P, applied normal to the surface of a semi-infinite, elastic, isotropic and homogenous mass (with elastic constant E and μ). The vertical normal stress, σ_z at any point as defined in Figure 2.19 is given by the Equation 2.6.

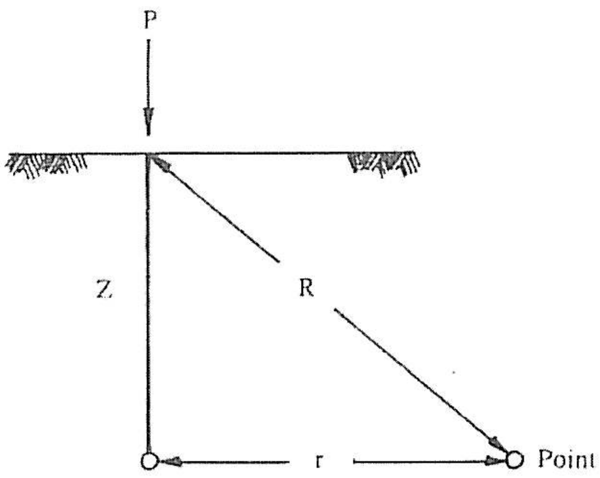


Figure 2.19: Definition sketch for Boussinesq's solution (after Rao, 1996)

$$\sigma_z = k\frac{P}{z^2} \tag{2.6}$$

where,

$$k = \frac{3}{2\pi}\left\{\frac{1}{1+(r/z)^2}\right\}^{\frac{5}{2}}$$

where,

r = radial distance from point load and

z = depth

The point load solution as above can be integrated suitably for obtaining the stress due to distributed load. In flexible pavement, the tyre contact imprint is essentially elliptical in shape, as seen from Fig 2.20. The variation in stresses follows the same general pattern as for point load case.

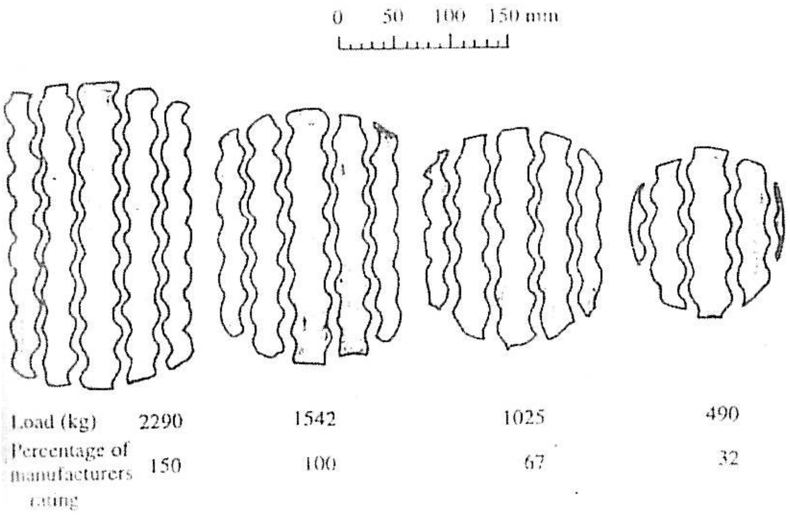


Figure 2.20: Tyre imprints of 8.25 x 20 tyre inflated to 480 kN/m^2 (after Rao, 1996)

The Boussinesq equation when integrated for uniformly loaded circular area gives the Equation 2.7 for vertical stress beneath the center.

$$\sigma_z = p_1 \left[1 - \frac{z^3}{(a^2 + z)^{\frac{3}{2}}} \right] \tag{2.7}$$

where,

a = radius of the circular area

p_l = uniformly distributed load

Figure 2.21 shows the variation of σ_z as pressure bulbs, which are same stress contours. The radial stress, σ_r is given by the Equation 2.8.

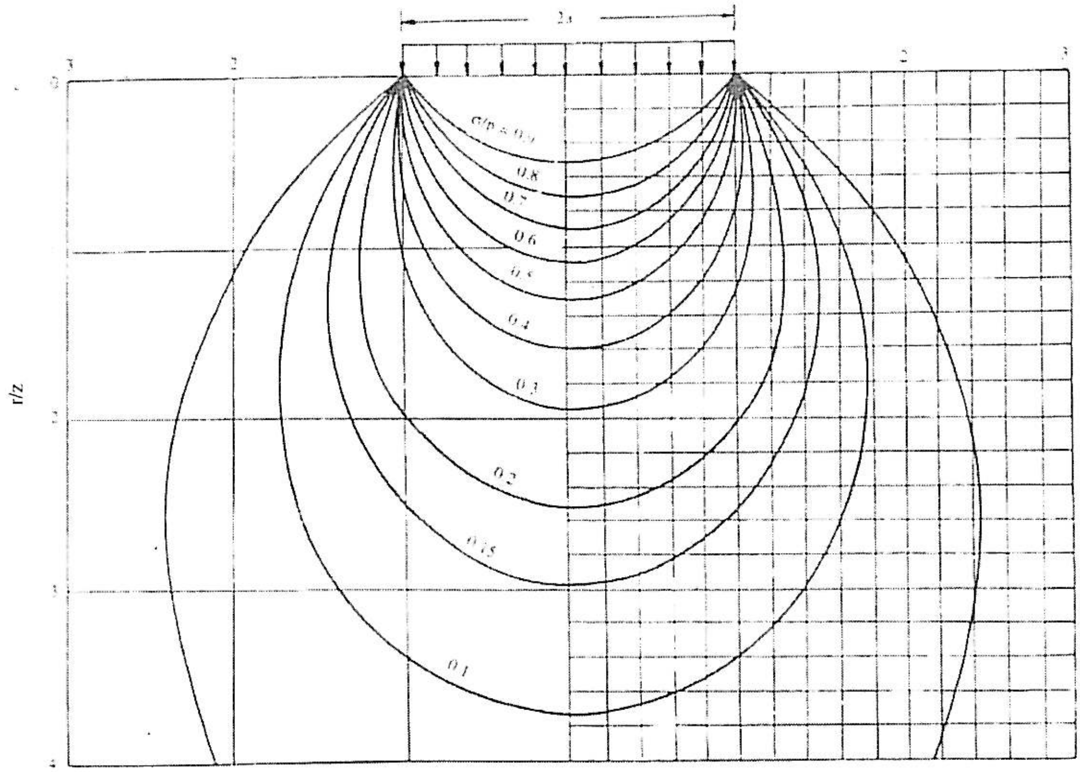


Figure 2:21 Vertical stresses due to uniform load on circular area (after Rao, 1996)

$$\sigma_r = \frac{P}{2}\left[1 + 2\mu - \frac{2(1+\mu)}{(a^2+z^2)^{\frac{1}{2}}} + \frac{z^3}{(a^2+z^2)^{\frac{3}{2}}}\right] \tag{2.8}$$

In any conventional flexible granular base/sub-base pavement structures having thin asphalt concrete surface course, it is assumed that the pavement portion does not contribute any deflection and it is only the sub-grade, which solely causes the total surface deflection as shown in the Equation 2.9.

$$\Delta T = \Delta P + \Delta S \approx \Delta S \qquad (2.9)$$

where,

ΔT = total surface deflection

ΔP = deflection within the pavement layer = 0

ΔS = deflection within the sub-grade

2.14 Road Loading

A pavement's function is to distribute the traffic load stresses to the soil (sub-grade) at a magnitude that will not shear or distort the soil. Typical soil-bearing capacities can be less than 345 kPa and in some cases as low as 14 to 21 kPa. Also, when soil is saturated with water, the bearing capacity can be very low, and in these cases it is very important for pavement to distribute tyre loads to the soil in such a way as to avoid failure of the pavement structure.

A typical automobile weighs approximately 12 kN, with tyre pressures of 240 kPa. These loads are small when compared to a typical tractor semi-trailer truck that can weigh up to 355.80 kN, the legal limit in many states, on five axles with tyre pressure of 690 kPa. Truckloads such as these represent the standard type of loading used in pavement design (Mannering and Kilareski, 1998).

Typical thickness of the individual layers for road is shown in Fig 2.22. These thickness vary with the type of axle loading, available materials and expected pavement design life, which is the number of years the pavement is expected to provide adequate service before it must undergo major rehabilitation.

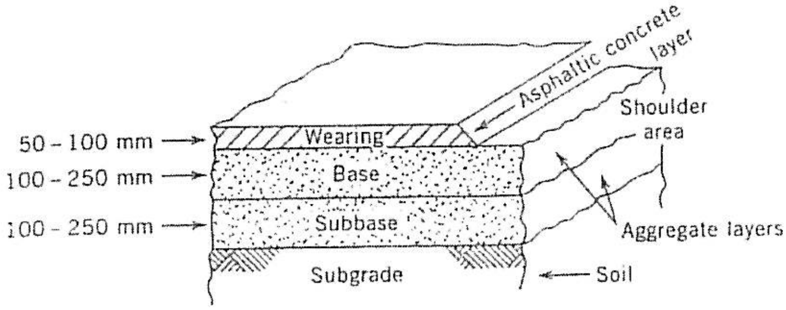


Figure 2.22: Typical flexible pavement cross section (after Mannering and Kilareski, 1998)

2.15 Calculation of Flexible Pavement Stresses and Deflections

To design a pavement structure, an engineer must be able to calculate the stresses and deflections in the pavement system. In the simplest case, the wheel load can be assumed to consist of a point load on a single-layer system, as shown in Figure 2.23. This type of load and configuration can be analyzed with the Boussinesq solutions that were derived for soil analysis. The Boussinesq theory assumes that the pavement is one layer thick and the material is elastic, homogenous and isotropic. Equation 2.10 gives the basic equation for stresses at a point in the system.

$$\sigma_z = 0.01K\frac{P}{z^2} \tag{2.10}$$

where,

σ_z is in kPa, P is the wheel load in newtons

z is the depth of the point in question in centimetres and K is a variable defined as

$$K = \frac{3}{2\pi\left[1 + (r/z)^2\right]^{\frac{5}{2}}}$$

where,

r is the radial distance in centimetres from the centreline of the point load to the point in question.

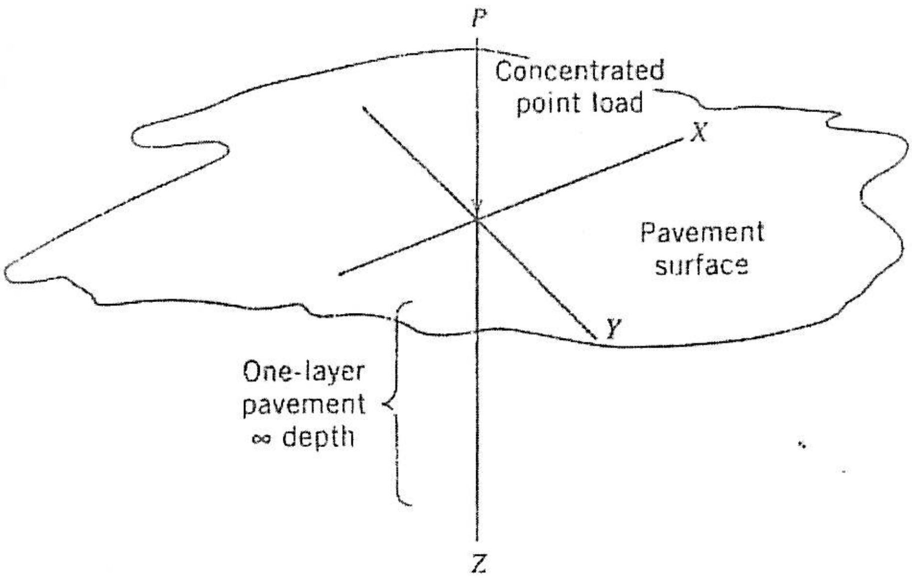


Figure 2.23: Point load on a one-layer pavement (after Mannering and Kilareski, 1998)

Although the Boussinesq theory is useful to begin the study of pavement stress calculations, it is not very representative of pavement-system loading and configuration because it applies to a point load on one layer. A more realistic approach is to expand the point load to an elliptical area that represents a tyre footprint. An equivalent circular area can define the tyre footprint with a radius calculated by the Equation 2.11.

$$a = \sqrt{\frac{P}{0.1 p_2 \pi}} \tag{2.11}$$

where,

P is the tyre load in newtons, p_2 is the tyre pressure in kPa
a is the equivalent load radius of the tyre footprint in centimetres.

The integration of the load from a point to a circular area can be used to determine the stresses and deflections in one-layer pavement system. As mentioned by Mannering and Kilareski (1998), that Ahlvin and Ulery provided solutions for the evaluation of stresses, strains and deflections at any point in a homogenous half-space. Their work makes it easier to analyze a more complex pavement system than that considered in the Boussinesq example. The Ahlvin and Ulery equation for the calculation of vertical stress, σ_z is shown in Equation 2.12.

$$\sigma_z = p\,(A + B) \qquad (2.12)$$

where,

p is the pressure due to the load in kPa
A and B are function values that depend on z/a and r/a, the depth in radii and offset distance in radii respectively.

The variables z, a, and r are as defined for the Boussinesq and circular load equations. The equation for radial-horizontal stress, σ_r (which is cause of pavement cracking) is shown in Equation 2.13.

$$\sigma_r = p(2\mu A + C + (1-2\mu)F) \qquad (2.13)$$

and the equation for deflection, Δ_z is given by Equation 2.14.

$$\Delta_z = \frac{p(1+\mu)a}{E}\left[\frac{z}{a}A + (1-\mu)H\right] \qquad (2.14)$$

where,

E is the modulus of elasticity (know as Young's modulus, the ratio of stress to strain as a load is applied to a material)

C, F and H are function values presented in Table 2.5.

Table 2.4: One-layer elastic function values

(after Mannering and Kilareski, 1998)

Function A																	
Depth (z/a)	Offset (r/a) 0	0.2	0.4	0.6	0.8	1	1.2	1.5	2	3	4	5	6	8	10	12	14
0	1.0	1.0	1.0	1.0	1.0	0.5	0	0	0	0	0	0	0	0	0	0	0
0.1	0.90050	0.89748	0.88679	0.86126	0.78797	0.43015	0.09645	0.02787	0.00856	0.00211	0.00084	0.00042					
0.2	0.80388	0.79824	0.77884	0.73483	0.63014	0.38269	0.15433	0.05251	0.01680	0.00419	0.00167	0.00083	0.00048	0.00020			
0.3	0.71265	0.70518	0.68316	0.62690	0.52081	0.34375	0.17964	0.07199	0.02440	0.00622	0.00250						
0.4	0.62861	0.62015	0.59241	0.53767	0.44329	0.31048	0.18709	0.09593	0.03118								
0.5	0.55279	0.54403	0.51622	0.46448	0.38390	0.28156	0.18556	0.09499	0.03701	0.01013	0.00407	0.00209	0.00118	0.0053	0.00025	0.00014	0.00009
0.6	0.48550	0.47691	0.45078	0.40427	0.33676	0.25588	0.17952	0.10010									
0.7	0.42654	0.41874	0.39491	0.35428	0.29833	0.21727	0.17124	0.10228	0.04538								
0.8	0.37531	0.36832	0.34729	0.31243	0.26581	0.21297	0.16206	0.10236									
0.9	0.33104	0.32492	0.30669	0.27707	0.23832	0.19488	0.15253	0.10094									
1	0.29289	0.28763	0.27005	0.24697	0.21468	0.17868	0.14329	0.09849	0.05185	0.01742	0.00761	0.00393	0.00226	0.00097	0.00050	0.00029	0.00018
1.2	0.23178	0.22795	0.21662	0.19890	0.17626	0.15101	0.12570	0.09192	0.05260	0.01935	0.00871	0.00459	0.00269	0.00115			
1.5	0.16795	0.16552	0.15877	0.14804	0.13436	0.11892	0.10296	0.08048	0.05116	0.02142	0.01013	0.00548	0.00325	0.00141	0.00073	0.00043	0.00027
2	0.10557	0.10453	0.10140	0.09647	0.09011	0.08269	0.07471	0.06275	0.04496	0.02221	0.01160	0.00659	0.00399	0.00180	0.00094	0.00056	0.00036
2.5	0.07152	0.07098	0.06947	0.06698	0.06373	0.05974	0.05555	0.04590	0.03787	0.02143	0.01221	0.00732	0.00463	0.00214	0.00115	0.00068	0.00043
3	0.05132	0.05101	0.05022	0.04886	0.04707	0.04487	0.04241	0.03839	0.03150	0.01980	0.01220	0.00770	0.00505	0.00242	0.00132	0.00079	0.00051
4	0.02986	0.02976	0.02907	0.02832	0.02802	0.02749	0.02651	0.02490	0.02193	0.01592	0.01109	0.00768	0.00536	0.00282	0.00160	0.00099	0.00065
5	0.01942	0.01938				0.01835			0.01573	0.01249	0.00947	0.00708	0.00527	0.00298	0.00179	0.00113	0.00075
6	0.01361					0.01307			0.01168	0.00983	0.00795	0.00628	0.00492	0.00299	0.00188	0.00124	0.00084
7	0.01005					0.00976			0.00894	0.00784	0.00661	0.00548	0.00445	0.00291	0.00193	0.00130	0.00091
8	0.00772					0.00755			0.00703	0.00635	0.00554	0.00472	0.00398	0.00276	0.00189	0.00134	0.00094
9	0.00612					0.00600			0.00566	0.00520	0.00466	0.00409	0.00353	0.00256	0.00184	0.00133	0.00096
10								0.00477	0.00465	0.00438	0.00397	0.00352	0.00326	0.00241			

Table 2.4 (Continued)

Function B																	
Depth (z/a)	Offset (r/a) 0	0.2	0.4	0.6	0.8	1	1.2	1.5	2	3	4	5	6	8	10	12	14
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.1	0.09852	0.10140	0.11138	0.13424	0.18796	0.05388	-0.07899	-0.02672	-0.00845	-0.00210	-0.00084	-0.00042					
0.2	0.16857	0.19306	0.20772	0.23524	0.25983	0.08513	-0.07759	-0.04443	-0.01593	-0.00412	-0.00166	-0.00083	-0.00024	-0.00010			
0.3	0.24362	0.26787	0.28018	0.29483	0.27257	0.10757	-0.04316	-0.04999	-0.02166	-0.00599	-0.00245						
0.4	0.32016	0.32259	0.32748	0.32273	0.26925	0.12404	-0.00766	-0.04535	-0.02522								
0.5	0.35777	0.35752	0.35323	0.33106	0.26236	0.13591	0.02165	-0.03455	-0.02651	-0.00991	-0.00388	-0.00199	-0.00116	-0.00049	-0.0025	-0.0014	-0.00009
0.6	0.37831	0.37531	0.36308	0.32822	0.25411	0.14440	0.04457	-0.02101									
0.7	0.38457	0.37962	0.36072	0.31929	0.24638	0.14986	0.06209	-0.00702	-0.02329								
0.8	0.38091	0.37408	0.35133	0.30699	0.23779	0.15292	0.07530	0.00614									
0.9	0.36962	0.36275	0.33734	0.29299	0.22891	0.15404	0.08507	0.01795									
1	0.35355	0.34553	0.32075	0.27819	0.21978	0.15355	0.09210	0.02814	-0.01005	-0.01115	-0.00608	-0.00344	-0.00210	-0.00092	-0.00048	-0.00028	-0.00018
1.2	0.31485	0.30730	0.28481	0.24836	0.20113	0.14915	0.10002	0.04378	0.00023	-0.00995	-0.00632	-0.00378	-0.00236	-0.00107			
1.5	0.25602	0.25025	0.23338	0.20694	0.17368	0.13732	0.10193	0.05745	0.01385	-0.00669	-0.00600	-0.00401	-0.00265	-0.00126	-0.00068	-0.00040	-0.00026
2	0.17889	0.18144	0.16644	0.15198	0.13375	0.11331	0.09254	0.06371	0.02836	0.00028	-0.00410	-0.00371	-0.00278	-0.00148	-0.00084	-0.00050	-0.00033
2.5	0.12807	0.12633	0.12126	0.11327	0.10298	0.09130	0.07869	0.06022	0.03429	0.00661	-0.00130	-0.00271	-0.00250	-0.00156	-0.00094	-0.00059	-0.00039
3	0.09487	0.09394	0.09099	0.08635	0.08033	0.07325	0.06551	0.05354	0.03511	0.01112	0.00157	-0.00134	-0.00192	-0.00151	-0.00099	-0.00065	-0.00046
4	0.05707	0.05666	0.05562	0.05383	0.05145	0.04773	0.04532	0.03995	0.03066	0.01515	0.00595	0.00155	-0.00029	-0.00109	-0.00094	-0.00068	-0.00050
5	0.03772	0.03760				0.03384			0.02474	0.01522	0.00810	0.00371	0.00132	-0.00043	-0.00070	-0.00068	-0.00049
6	0.02666					0.02468			0.01968	0.01380	0.00867	0.00496	0.00254	0.00028	-0.00037	-0.00047	-0.00045
7	0.01980					0.01868			0.01577	0.01204	0.00842	0.00547	0.00332	0.00093	-0.00002	-0.00029	0.00037
8	0.01526					0.01459			0.01279	0.01034	0.00779	0.00554	0.00372	0.00141	0.00035	-0.00008	-0.00025
9	0.01212					0.01170			0.01054	0.00888	0.00705	0.00533	0.00386	0.00178	0.00066	0.00012	-0.00012
10								0.00924	0.00879	0.00764	0.00631	0.00501	0.00382	0.00199			

Table 2.4 (Continued)

	Function C																
Depth	Offset (r/a)																
(z/a)	0	0.2	0.4	0.6	0.8	1	1.2	1.5	2	3	4	5	6	8	10	12	14
0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
0.1	-0.04926	-0.05142	-0.05903	-0.07708	-0.12108	0.02247	0.12007	0.04475	0.01536	0.00403	0.00164	0.00082					
0.2	-0.09429	-0.09755	-0.10872	-0.12977	-0.14552	0.02419	0.14896	0.07892	0.02951	0.00796	0.00325	0.00164	0.00094	0.00039			
0.3	-0.13181	-0.13434	-0.14415	-0.15023	-0.12990	0.01988	0.13394	0.09816	0.04148	0.01163	0.00483						
0.4	-0.16008	-0.16188	-0.16519	-0.15985	-0.11168	0.01292	0.11014	0.10422	0.05067								
0.5	-0.17889	-0.17835	-0.17497	-0.15625	-0.09833	0.00483	0.08730	0.10125	0.05690	0.01824	0.00778	0.00399	0.00231	0.00098	0.00050	0.00029	0.00018
0.6	-0.18915	-0.18633	-0.17336	-0.14934	-0.08967	-0.00304	0.06731	0.09313									
0.7	-0.19244	-0.18831	-0.17393	-0.14147	-0.08409	-0.01061	0.05028	0.08253	0.06129								
0.8	-0.19046	-0.18481	-0.16784	-0.13393	-0.08066	-0.01744	0.03582	0.07114									
0.9	-0.18481	-0.17841	-0.16024	-0.12664	-0.07828	-0.02337	0.02359	0.05993									
1	-0.17678	-0.17050	-0.15188	-0.11975	-0.07634	-0.02843	0.01331	0.04939	0.05429	0.02726	0.01333	0.00726	0.00433	0.00188	0.00098	0.00057	0.00036
1.2	-0.15742	-0.15117	-0.13467	-0.10763	-0.07289	-0.03575	-0.00245	0.03107	0.04522	0.02791	0.01467	0.00824	0.00501	0.00221			
1.5	-0.12801	-0.12277	-0.11101	-0.09145	-0.06711	-0.04124	-0.01702	0.01088	0.03154	0.02652	0.01570	0.00931	0.00585	0.00266	0.00141	0.00083	0.00039
2	-0.08944	-0.08491	-0.07976	-0.06925	-0.05560	-0.04144	-0.02687	-0.00782	0.01267	0.02070	0.01527	0.01013	0.00321	0.00327	0.00179	0.00107	0.00069
2.5	-0.06403	-0.06068	-0.05839	-0.05259	-0.04522	-0.03605	-0.02800	-0.01536	0.00103	0.0134	0.00987	0.00707	0.00569	0.00209	0.00128	0.00083	
3	-0.04744	-0.04560	-0.04339	-0.04089	-0.03642	-0.03130	-0.02587	-0.01748	-0.00528	0.00792	0.01030	0.00888	0.00689	0.00392	0.00232	0.00145	0.00096
4	-0.02854	-0.02737	-0.02562	-0.02585	-0.02421	-0.02112	-0.01964	-0.01586	-0.00956	0.00038	0.00492	0.00602	0.00561	0.00389	0.00254	0.00168	0.00155
5	0.01886	-0.01810				-0.01568			-0.00939	-0.00293	-0.00128	0.00329	0.00391	0.00341	0.00250	0.00177	0.00127
6	-0.01333					-0.01118			-0.00819	-0.00405	-0.00079	0.00129	0.00234	0.00272	0.00227	0.00173	0.00130
7	-0.00990					-0.00902			-0.00678	-0.00417	-0.00180	-0.00004	0.00113	0.00200	0.00193	0.00161	0.00128
8	-0.00763					-0.00679			-0.00552	-0.00393	-0.00225	-0.00077	0.00029	0.00134	0.00157	0.00143	0.00120
9	-0.00607					-0.00423			-0.00452	-0.00353	-0.00235	-0.00118	-0.00027	0.00082	0.00124	0.00122	0.00110
10								-0.00381	-0.00373	-0.00314	-0.00233	-0.00137	-0.00630	0.00040			

Table 2.4 (Continued)

	Function F																
Depth	Offset (r/a)																
(z/a)	0	0.2	0.4	0.6	0.8	1	1.2	1.5	2	3	4	5	6	8	10	12	14
0	0.5	0.5	0.5	0.5	0.5	0	-0.34722	-0.22222	-0.12500	-0.05556	-0.03125	-0.02000	-0.01389	-0.00781	-0.00500	-0.00347	-0.00255
0.1	0.45025	0.44794	0.43981	0.41954	0.35789	0.03817	-0.20800	-0.17612	-0.10950	-0.05151	-0.02961	-0.01917					
0.2	0.40194	0.39781	0.38294	0.34823	0.26215	0.05466	-0.11165	-0.13381	-0.09441	-0.04750	-0.02798	-0.01835	-0.01295	-0.00742			
0.3	0.35633	0.35094	0.34508	0.29016	0.20503	0.06372	-0.05346	-0.09768	-0.08010	-0.04356	-0.02636						
0.4	0.31431	0.30801	0.28681	0.24469	0.17086	0.06848	-0.01818	-0.06835	-0.06684								
0.5	0.27639	0.26997	0.24890	0.20937	0.14752	0.07037	0.00388	-0.04529	-0.05479	-0.03595	-0.02320	-0.01590	-0.00154	-0.00681	-0.00450	-0.00318	-0.00237
0.6	0.24275	0.23444	0.21667	0.18138	0.13042	0.07068	0.01797	-0.02474									
0.7	0.21327	0.20762	0.18956	0.15903	0.11740	0.06963	0.02704	-0.01392	-0.03469								
0.8	0.18765	0.18287	0.16679	0.14053	0.10604	0.06774	0.03277	-0.00365									
0.9	0.16552	0.16158	0.14747	0.12528	0.09664	0.06533	0.03619	0.00408									
1	0.14645	0.14280	0.12395	0.11225	0.08850	0.06256	0.03819	0.00984	-0.01367	-0.01994	-0.01591	-0.01209	-0.00931	-0.00587	-0.00400	-0.00289	-0.00219
1.2	0.11589	0.11360	0.10460	0.09449	0.07486	0.05670	0.03913	0.01716	-0.00452	-0.01491	-0.01337	-0.01068	-0.00844	-0.00550			
1.5	0.08398	0.08196	0.07719	0.06918	0.05919	0.04804	0.03686	0.02177	0.00413	-0.00879	-0.00995	-0.00870	-0.00723	-0.00495	-0.00353	-0.00261	-0.00201
2	0.05279	0.05348	0.04934	0.04614	0.04162	0.03593	0.03029	0.02197	0.01043	-0.00189	-0.00546	-0.00589	-0.00544	-0.00410	-0.00307	-0.00233	-0.00183
2.5	0.03576	0.03673	0.03459	0.03263	0.03014	0.02762	0.02406	0.01927	0.01188	0.00198	-0.00226	-0.00364	-0.00386	-0.00332	-0.00263	-0.00208	-0.00166
3	0.02566	0.02586	0.02255	0.02575	0.02253	0.02097	0.01511	0.01623	0.01144	0.00396	-0.00010	-0.00192	-0.00258	-0.00263	-0.00223	-0.00183	-0.00150
4	0.01493	0.01536	0.01412	0.01259	0.01366	0.01331	0.01256	0.01134	0.00912	0.00508	0.00209	0.00026	-0.00076	-0.00148	-0.00153	-0.00137	-0.00120
5	0.00871	0.01011				0.00905			0.00700	0.00475	0.00277	0.00129	0.00031	-0.00066	-0.00096	-0.00099	-0.00093
6	0.00680					0.00675			0.00538	0.00409	0.00278	0.00170	0.00088	-0.00013	-0.00053	-0.00066	-0.00070
7	0.00503					0.00483			0.00428	0.00346	0.00258	0.00178	0.00114	0.00027	-0.00020	-0.00041	-0.00049
8	0.00386					0.00380			0.00350	0.00291	0.00229	0.00174	0.00125	0.00048	0.00003	-0.00020	-0.00033
9	0.00306					0.00374			0.00291	0.00247	0.00203	0.00163	0.00124	0.00062	0.00020	-0.00005	-0.00019
10								0.00267	0.00246	0.00213	0.00176	0.00149	0.00126	0.00070			

Table 2.4: (Continued)

Depth (z/a)	Function H																
	Offset (r/a)																
	0	0.2	0.4	0.6	0.8	1	1.2	1.5	2	3	4	5	6	8	10	12	14
0	2.0	1.97987	1.91751	1.80575	1.62553	1.27319	0.93676	0.71185	0.51671	0.33815	0.25200	0.20045	0.15626	0.12576	0.09518	0.08346	0.07023
0.1	1.80998	1.79018	1.72886	1.61961	1.44711	1.18107	0.92670	0.70888	0.51627	0.33794	0.25184	0.20081					
0.2	1.63961	1.62068	1.56242	1.46001	1.30614	1.07996	0.90098	0.70074	0.51382	0.33726	0.25162	0.20072	0.15688	0.12512			
0.3	1.48806	1.47044	1.40979	1.32442	1.19210	1.02740	0.86726	0.68823	0.50966	0.33638	0.25124						
0.4	1.35407	1.33802	1.28963	1.20822	1.09555	0.96202	0.83042	0.67238	0.50412								
0.5	1.23607	1.22176	1.17894	1.10830	1.01312	0.90298	0.79308	0.65429	0.49278	0.33293	0.24996	0.19982	0.16668	0.12493	0.09996	0.08295	0.07123
0.6	1.13238	1.11938	1.08350	1.02154	0.94120	0.84917	0.75653	0.63469									
0.7	1.04131	1.03037	0.99794	0.91049	0.87742	0.80030	0.72143	0.61442	0.48061								
0.8	0.96125	0.95175	0.92386	0.87928	0.82136	0.75571	0.68809	0.59398									
0.9	0.89072	0.88251	0.85856	0.82616	0.77950	0.71495	0.65677	0.57361									
1	0.82843	0.85005	0.80465	0.76809	0.72587	0.67769	0.62701	0.55364	0.45122	0.31877	0.24386	0.19673	0.16516	0.12394	0.09952	0.08292	0.07104
1.2	0.72410	0.71882	0.70370	0.67937	0.64814	0.61187	0.57329	0.51552	0.43013	0.31162	0.24070	0.19520	0.16369	0.12350			
1.5	0.60555	0.60233	0.57246	0.57633	0.55559	0.53138	0.50496	0.46379	0.39872	0.29945	0.23495	0.19053	0.16199	0.12281	0.09876	0.08270	0.07064
2	0.47214	0.47022	0.44512	0.45656	0.44502	0.43202	0.41702	0.39242	0.35054	0.27740	0.22418	0.18618	0.15846	0.12124	0.09792	0.08196	0.07026
2.5	0.38518	0.38403	0.38098	0.37608	0.36940	0.36155	0.35243	0.33698	0.30013	0.25550	0.21208	0.17898	0.15395	0.11928	0.09700	0.08115	0.06980
3	0.32457	0.32403	0.32184	0.31587	0.31464	0.30969	0.30381	0.29364	0.27453	0.23487	0.19977	0.17154	0.14919	0.11694	0.09558	0.08061	0.06897
4	0.24620	0.24588	0.24820	0.25128	0.24168	0.23932	0.23668	0.23164	0.22188	0.19908	0.17640	0.15596	0.13864	0.11172	0.09300	0.07864	0.06848
5	0.19805	0.19785				0.19455			0.18450	0.17080	0.15575	0.14130	0.12785	0.10585	0.08915	0.07675	0.06695
6	0.16554					0.16326			0.15750	0.14868	0.13842	0.12792	0.11778	0.09990	0.08562	0.07452	0.06522
7	0.14217					0.14077			0.13699	0.13097	0.12404	0.11620	0.10843	0.09387	0.08197	0.07210	0.06377
8	0.12448					0.12352			0.12112	0.11680	0.11176	0.10600	0.09976	0.08848	0.07800	0.06928	0.06200
9	0.11079					0.10989			0.10854	0.10548	0.10161	0.09702	0.09234	0.08298	0.07407	0.06678	0.05976
10								0.9900	0.09820	0.09510	0.09290	0.08980	0.08300	0.07710			

The Boussinesq theory and Alvin and Ulery equations can be used to calculate the stresses and deflections in a simple pavement system. The utility of these is that they serve as a basis for more complex pavement analysis.

2.16 Pavement Design Method

There are four basic methods of pavement design, each of which has its merits and advocates. They are:

a. Design based on experience

b. Design using a combination of field experience and laboratory tests

c. Design based on equivalence

d. Design based on mathematical models

2.16.1 Design Based on Experience

The finding of a close match between two pavements and the fact that reliable construction and maintenance records are rarely available means that this method of design has to be conservative, so although it may allow relatively cheap design it does not always result in the cheapest pavement.

This form of design has merits when undertaking the construction of minor pavements, for which the costs of design will outweigh any potential savings that could be achieved from making a detailed site investigation and critical design.

2.16.2 Design Using a Combination of Field Experience and Laboratory Tests

This has been the basic method of design for public highways in the UK for many years, largely relying on full-scale trials made by Transport and Road Research Laboratory (TRRL), the government-sponsored highway research laboratory and the experience of local authority highway engineers.

2.16.3 Design Based on Equivalence

This method of design has been used to allow the knowledge gained from previous experience of full-scale trials to be applied to paving materials of which there is no previous experience. Their reliability depends upon the accuracy of the determination of the equivalence, normally by laboratory tests and may overlook some long-term (benefit or dis-benefit) of the new material.

2.16.4 Design Based on Mathematical Model

In common with other structures, a pavement may be designed entirely by mathematical analysis. This was not possible until there was a wide availability of powerful computers, as there were too many variables for practical manual analysis. These techniques are very useful in developing an understanding of the essential problem of pavement design, but their reliability depends on the accuracy of the modeling, which is not sufficiently understood at present.

The technique of design using a combination of field experience and laboratory tests allows the application of reliable practical experience. The results of full-scale experiments ensure that all factors affecting the performance of the pavement have been accurately measured and their variability quantified. On the other hand, the experience gain by studying the performance of as-built existing road networks is also utilized in the design of pavements (Road Note 31).

2.17 The Weight Limit Design Concept for Lime Stabilized Capping Layer

Figure 2.24 shows the general concept of the model used in this research in order to study the behaviour of lime stabilized soil as a capping layer in road construction. The load from the vehicles will be transferred from the pavement through the road base, sub-base and finally to the sub-grade. In this regard, the sub-grade must be good enough to sustain the transferred load to avoid any failure. If the sub-grade is weak, various methods are employed to strengthen it in order to take vehicular load. Normally, in Malaysia, the weak or marginal sub-grade is removed and replaced with granular materials which serve as capping layers for the subsequent construction of the road (Jabatan Kerja Raya, 1988).

The very task of strengthening the sub-grade is the main focus of this research. This research seeks to stabilize the weak sub-grade with lime instead of removing or replacing the sub-grade with other materials. The lime stabilized soil will be used as a capping layer for road construction and its behaviour under vehicular load is studied in this research as depicted in Figure 2.24. Consequently, by considering the various geotechnical properties and parameters of the lime stabilized capping layer, a weight limit of the road design is developed as an aid to the practicing pavement engineer.

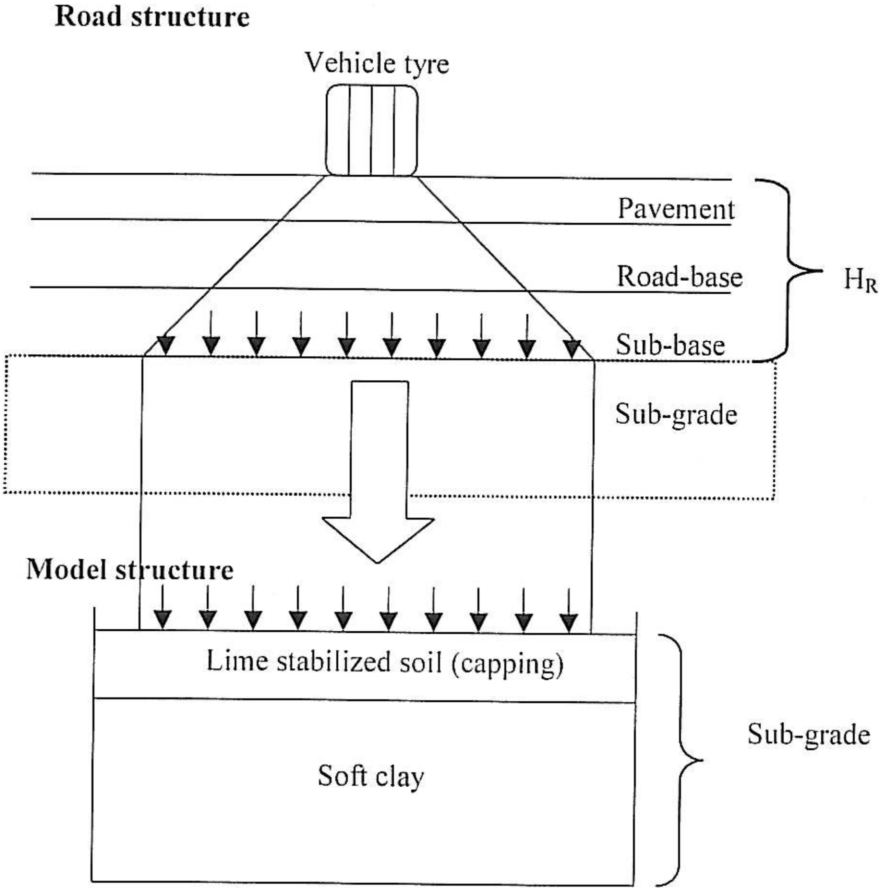


Figure 2.24: General concept for the model testing of the lime stabilized soils

CHAPTER III

TESTING EQUIPMENT AND PROCEDURES

3.1 Introduction

The testing in this research was conducted based on the references from the books, journal and standards. It was started with the collection of the samples from the different sites. Then, the properties testing were conducted for soils and lime to look their suitability before stabilization process. The strength of the unstabilized samples with the different moisture content was determined to get the correlation of the strength and moisture content. From this result, the soil-lime mixture was prepared with the moisture content based on the minimum strength of soil. Strength development test of the soil-lime mixture was conducted with the different ages of the curing period. The physical model was developed to construct the lime-stabilized capping layer underlain by soft clay layer to represent actual condition in field. Bearing capacity of the lime-stabilized capping layer underlain soft clay layer is measured. From the bearing capacity of the structure, the design of weight limit of the road which not overstressed the sub-grade is suggested.

3.2 Preliminary Test on the Unstabilized Materials

The following laboratory testing was carried out in accordance with BS 1377: Part1, Part 2 and Part 3: 1990. These tests are essential to determine the basic properties of unstabilized materials. Knowledge of these properties basically is to ensure that material is in accordance with limits given in the specifications. Also, a good understanding of changes in these properties is important to establish the correlation due to the addition of additive in soil.

3.2.1 Sampling

All of the soil samples were obtained from three different locations around Johor Bahru and Kulai. First sample was obtained from Tanjung Pelepas, known as Tanjung Pelepas Marine Clay (Figure 3.1). Next two samples were obtained at Universiti Teknologi Malaysia as UTM Yellowish Clay and at Kulai, Johor as Kulai Pinkish White Clay (Figure 3.2 and 3.3).

All samples collected were placed and kept in the covered tank and properly labeled. Representative samples were taken to study their base properties according to BS1924: Part 1: 1990.

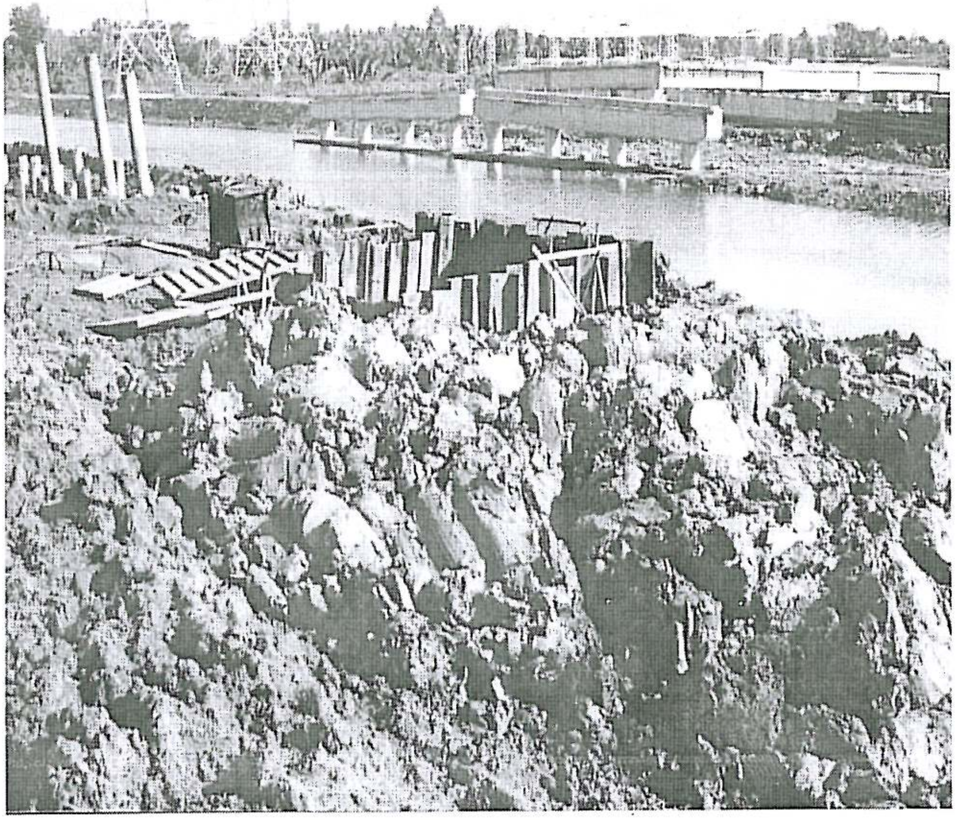


Figure 3.1: The location of collected soil sample namely Tg. Pelepas Marine Clay

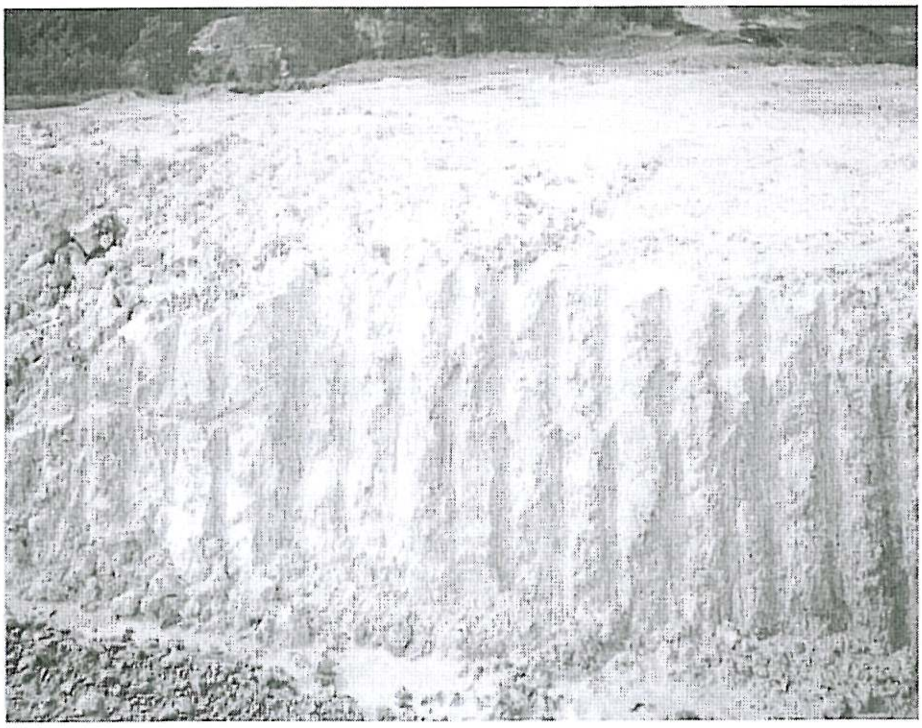


Figure 3.2: The location of collected soil sample namely UTM Yellowish Clay

Figure 3.3: The location of collected soil sample namely Kulai Pinkish White Clay

3.2.2 Particle Density

Particle density values were obtained from specific gravity test based on BS1377: Part 2: 1990, Clause 8.3 using small pyknometer, which is suitable for those soil consisting clay particles. This method is suitable for soils consisting of particles finer than 2 mm. Larger particles may be ground down to smaller than this size before testing.

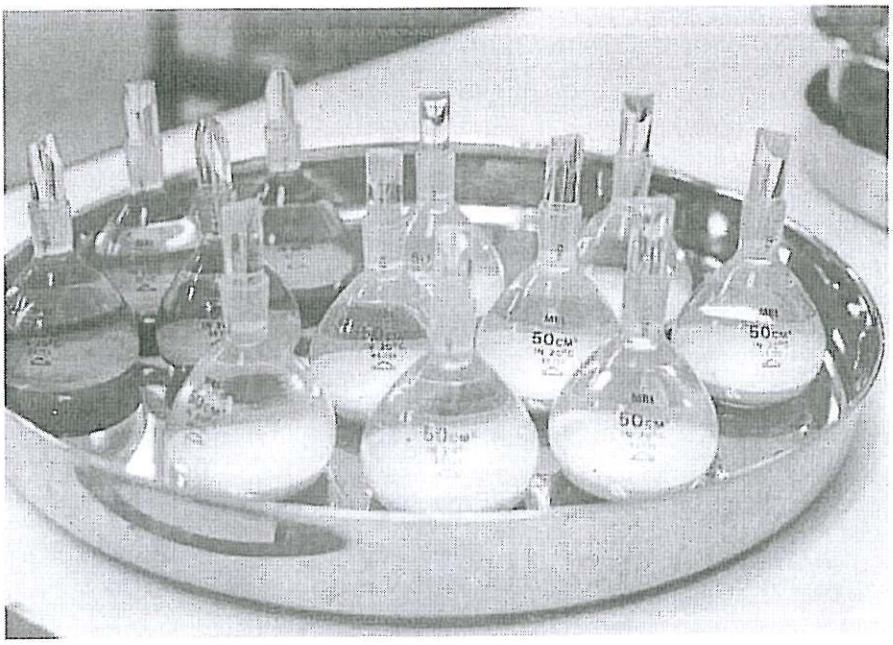


Figure 3.4: Small pyknometer bottle for particle density test

3.2.3 Particle Size Distribution

The particle size distribution is required to ensure that the soils for stabilization are in accordance with the limits given in the specifications for lime-stabilized materials. Two methods of sieving are specified with wet sieving (Figure 3.5) as the definitive method of applicable to essentially cohesive soil (BS 1377: Part 2: 1990, Clause 9.2) while dry sieving (Figure 3.6), is suitable only for soils containing insignificant quantities of silt and clay (BS 1377: Part 2: 1990, Clause 9.3). The determination of the size distribution of fine particles down to clay size, sedimentation methods are specified and in this research, hydrometer sedimentation method was used (BS 1377: Part 2: 1990, Clause 9.5) as illustrated in Figure 3.7. The combination of sieving and sedimentation results enables continuous particles size distribution curve of a soil from the coarsest particle down to clay size.

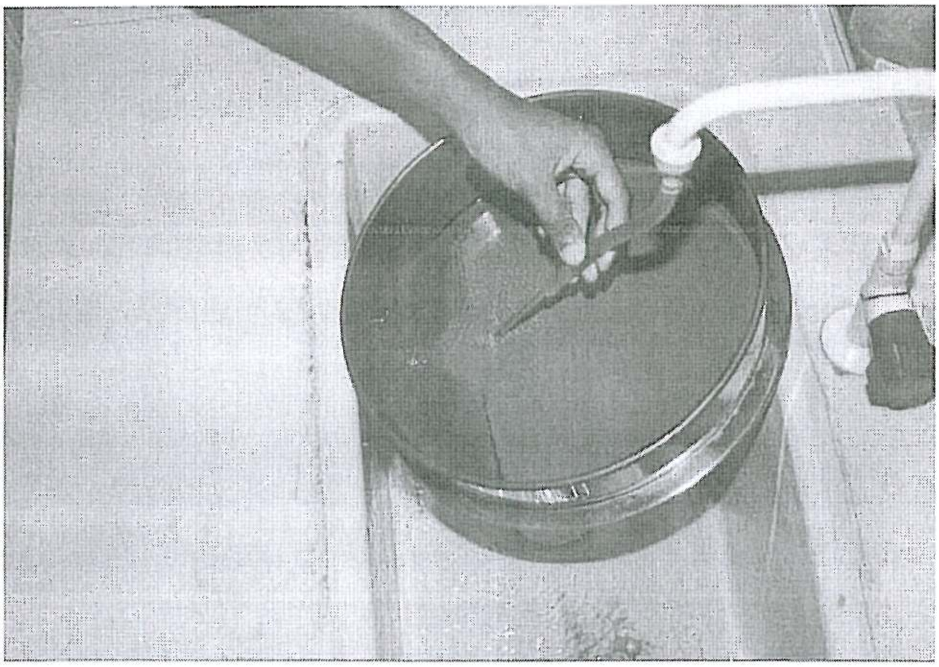


Figure 3.5: Wet sieving method

3.2.4 Soil Classification

For engineering purposes, soils are frequently classified into groups. Two common classification systems are Unified Soil Classification System (USCS), used for generally engineering purposes, and the system developed by the American Association of State Highway and Transportation Officials and often used for soils in highway engineering. Other systems have been developed, but these two are the most commonly used system in North America. The American Society for Testing and Materials

(ASTM) has published the Unified System as a Standard D2487 and illustrated in Figure 3.8 (a) (Atkins, 2003). Beside that, British Standard Institution (BSI) also proposed the soil classification chart, which provide five range of liquid limit as shown in Figure 3.8 (b).

Figure 3.6: Dry sieving test apparatus

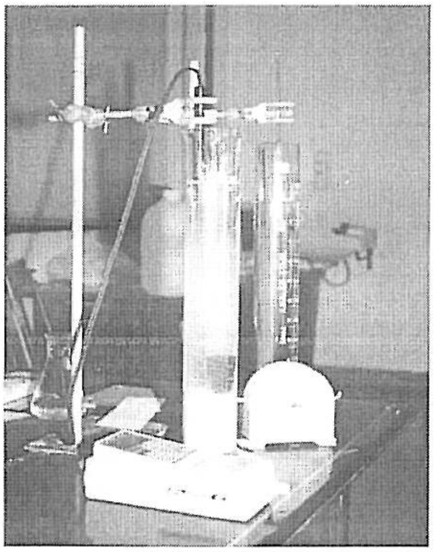
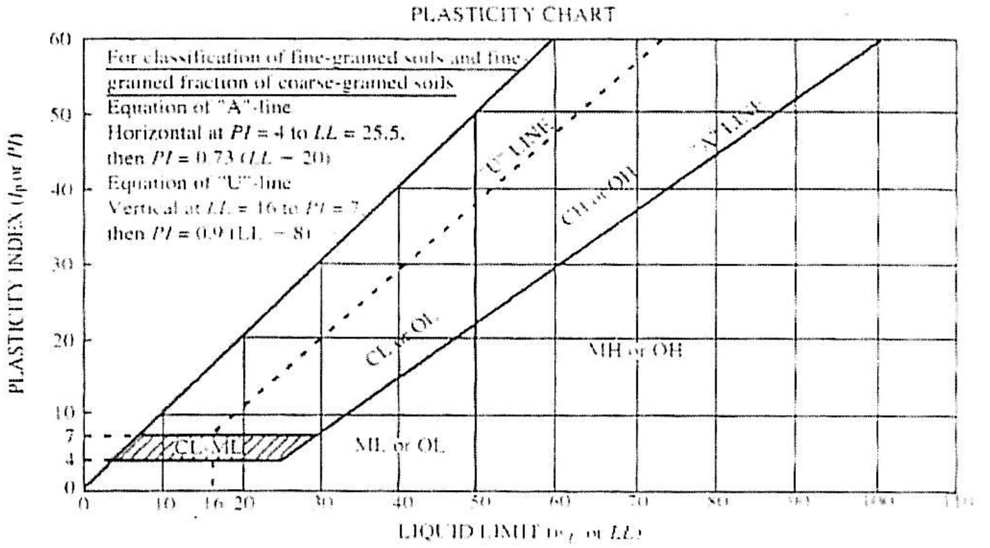
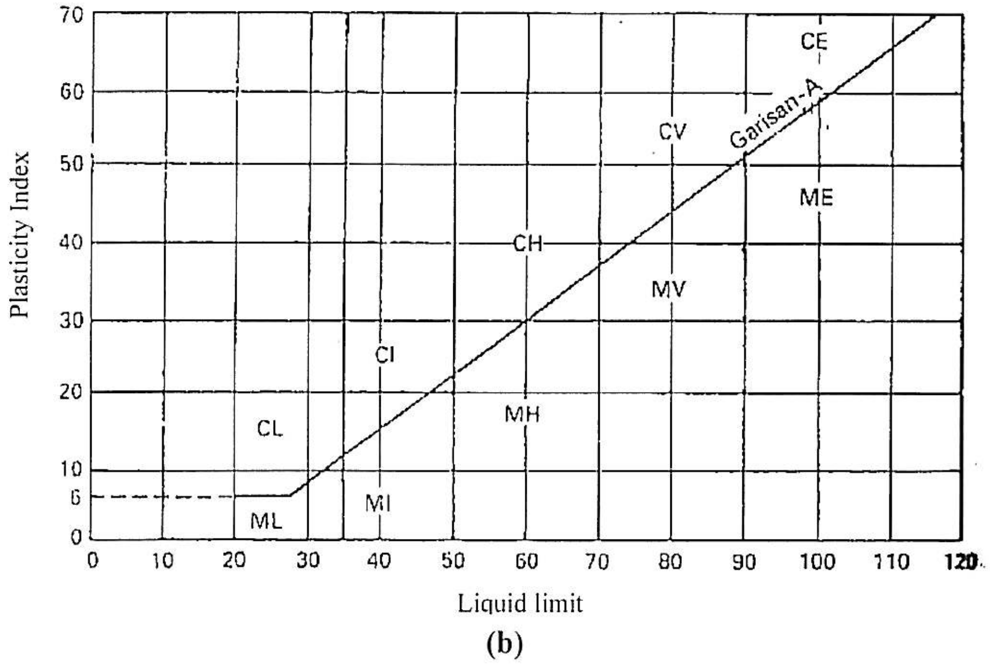


Figure 3.7: Hydrometer sedimentation method apparatus



Note: The "U" line represents approximate upper limits of w_l and I_p for natural soils. The "A" line separates the silt (M) and clay (C) soils. Fine-grained soils with plasticity values in the hatched area are classified as CL-ML.

(a)



(b)

Figure 3.8: Soil classification chart (a) Unified Soil Classification System (USCS) (after Atkins, 2003) (b) BS 5930: 1981 (after Aminaton Marto et.al, 1993)

3.2.5 Atterberg Limits

A material with a plasticity index lower than 10 per cent may not contain sufficient reactive materials for it to be stabilized with lime. Conversely, a material with a plasticity index above 25 per cent is suitable for lime stabilization. Knowledge of the plasticity index of a material is thus required for lime stabilization. The procedure for determining the plasticity index of materials to be used for lime stabilization is described in BS 1377: Part 2: 1990, Clause 4 and 5. The determination of liquid limit with cone penetrometer method was used in this research and the appropriate apparatus is shown in Figure 3.9. This method covers the determination of liquid limit of a sample of soil in its natural state or of a sample of soils from which material retained on a 425 μm test sieve has been removed. The liquid limit is the value of the moisture content corresponding to a cone penetration of 20 mm to one decimal place.

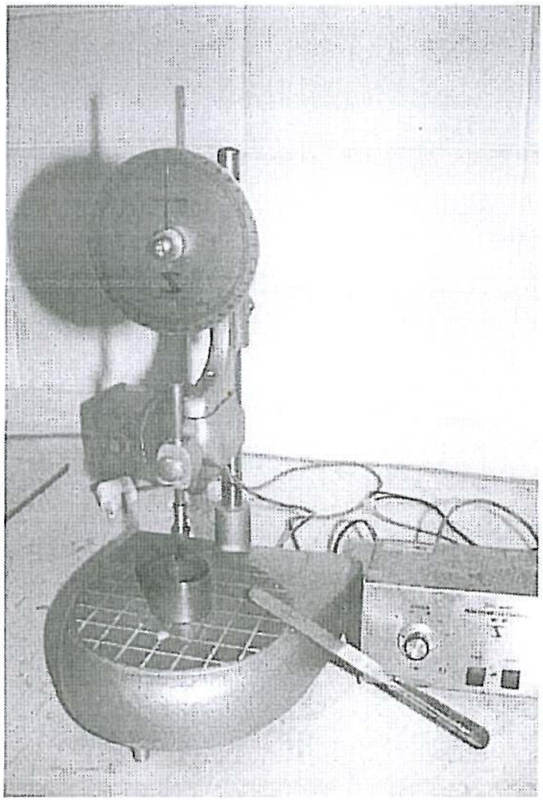


Figure 3.9: Liquid limit test (Cone penetrometer apparatus)

The plastic limit is the empirically established moisture content at which soil starts to behave like plastic. The soil shall not to be allowed to become dry before testing. It is convenient to carry out the test on portions of the material prepared for one of the liquid limit test procedures. The plastic limit is obtained from the moisture content at the soil show the shears both longitudinally and transversely when it has been rolled to about 3 mm diameter. It used together with liquid limit to determine the index of plasticity of a soil, PI, using the Equation 3.1.

$$\text{Index of plasticity (PI)} = \text{LL} - \text{PL} \qquad (3.1)$$

3.3 Preliminary Test on the Stabilizer

All commercial lime products are likely to have impurities (carbonate, silica, alumina, etc.), which dilute the active additive but are not harmful to the stabilization reaction. In this research, hydrated lime, sometimes also called as slaked lime was used for the purpose of the stabilization process. Hydrated lime can be obtained in the bag as shown in Figure 3.10.

Figure 3.10: 25 kg packaging of hydrated lime

3.3.1 Suitability of Lime

This test determines the suitability of lime for the purpose of lime stabilization in accordance with BS 1924: Part 2: 1990, Clause 5.4.6. The quality of lime being used must satisfy the pH value of 12.35 to 12.45 at 25°C or else the lime is not suitable. A saturated solution of calcium hydroxide was prepared by placing about 5 g of calcium hydroxide and 100 mL of CO_2 – free distilled water into the shaking machine. The pH value of the solution was determined by using the calibrated pH meter (**APPENDIX A**) as shown in Figure 3.11 to the 0.05 units and its temperature was recorded to the nearest 0.5°C. The pH value of the solution was then corrected at 25°C using the Equation 3.2.

$$pH_{25} = pH_T + 0.03\ (T - 25) \qquad (3.2)$$

where,

pH_T is the pH at measured temperature, T°C

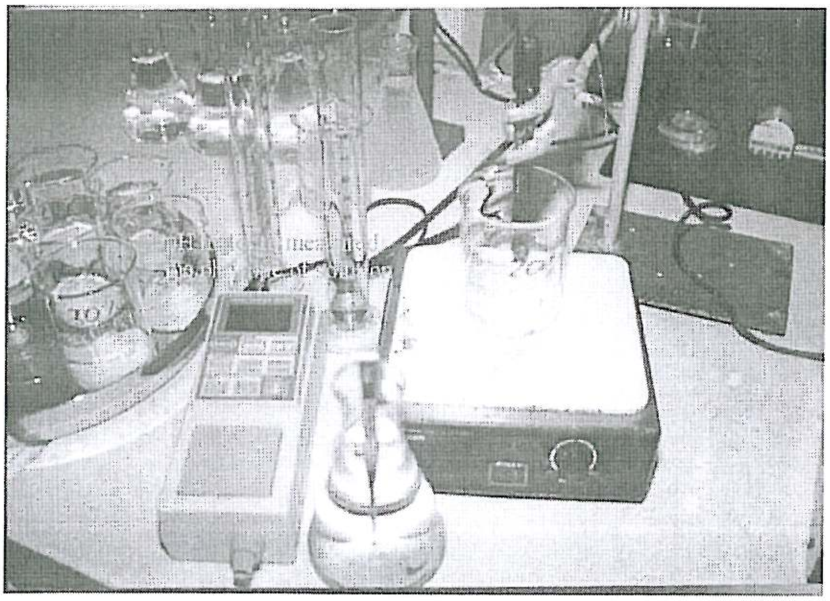


Figure 3.11: pH meter to measure the pH value of the saturated calcium hydroxide solution during the determination of the Initial Consumption of Lime (ICL) test

3.3.2 Available Lime Content

It was determined by using the sucrose extraction method based on BS 6463: Part 2: 1984, Clause 20. The content of the calcium oxide and calcium hydroxide present in lime was determined by shaking the lime sample with a solution of sucrose. After the residue has been filtered off, the solution was titrated against standard hydrochloric acid using phenolphthalein as an indicator and the titration process was stopped when the colour of the solution changed from faint pink to pale yellowish white. The volume of hydrochloric acid used in the titration process was calculated in terms of calcium oxide or calcium hydroxide by the following Equation 3.3 and 3.4 (BS 6463: Part 2: 1984: Clause 20.3).

$$\text{Percentage available lime (as CaO)} = \frac{2.804V}{m} \quad (3.3)$$

$$\text{Percentage available lime (as Ca(OH)}_2\text{)} = \frac{3.705V}{m} \quad (3.4)$$

where,

V - is the titration in mL

m - is the mass of sample (in gram, g)

3.3.3 Determination of the Initial Consumption of Lime (ICL)

The purpose of the test is to determine the amount of lime that is needed to achieve significant improvement in the properties of the soil and its take about one hour to complete. As a result, the amount of lime to ensure that a pH of 12.4 is reached in order to sustain the strength development produced from lime-pozzolanic reaction was obtained. The test procedure was followed accordance to BS 1924: Part 2: 1990, Clause 5.4. Samples of the material were mixed with water and different proportion of lime being used. The saturated solutions were then measured for their pH values using the pH meter. The apparatus of the ICL test is shown in Figure 3.11.

The basic concept of the test was based on the fact that a saturated solution of lime (calcium hydroxide) in distilled water, which free of carbon dioxide, give a pH value of 12.4 at 25°C. This situation required maintaining the reaction between lime and reactive components in the material to be stabilized. As an expression, ICL of the material was obtained by the minimum amount of lime needed to give a pH value of 12.40 at 25°C.

3.4 Remoulded Soil Strength

The remoulded strength of the unstabilized soils was determined with the California Bearing Ratio (CBR), Unconfined Compressive Strength (UCS) and laboratory Vane Shear Strength test. Each samples was tested with the different moisture content to get the correlation between the strength and water content of the soil. Basically, the strength of the cohesive soil will decrease when the moisture content increases and this soil is not suitable for the engineering purposes. For the purpose of the road construction, the lime-stabilized method was suggested to increase the strength of the cohesive soil. According to the Manual on Pavement Design, Arahan Teknik Jalan 5/85, 1985 the total flexible pavement thickness was based on the required Corrected Equivalent Thickness, T_A', which obtained from the thickness design nomograph. From the nomograph, the minimum design value of the California Bearing Ratio (CBR) for the sub-grade is 2 %. Therefore, the work for the lime stabilization strength development will be based on the CBR value of 2 % for each soils sample.

3.4.1 California Bearing Ratio (CBR) Test

The test was conducted according to BS 1377: Part 4: 1990, Clause 7. The principle of the test is to determine the relationship between force and penetration value when a cylindrical plunger penetrate the soil to specified depth. At certain values of penetration, the ratio of the applied force to standard force, expressed as percentage is defined as the California Bearing Ratio (CBR).

3.4.1.1 Sample Preparation

Generally the soil strength varies with its natural moisture content. Increasing of moisture content value will result in the decreasing of the soil strength. In order to have a wide range of soil strength, the CBR samples were prepared at different value of moisture content. In this research, Tg. Pelepas Marine Clay samples were prepared at moisture content ranging from 8 % to 50 %. For UTM Yellowish Clay, the moisture content ranges from 10 % to 60 % and Kulai Pinkish White Clay was prepared at moisture content ranges from 10 % to 80%.

The sample was compacted in the cylindrical, corrosion-resistant, metal CBR mould, having a nominal diameter of 152 ± 0.5 mm with a detachable base-plate and a removal collar extension. A metal 4.5 kg rammer was used to compact the sample. All of the apparatus for the CBR sample preparation are as shown in Figure 3.12. The soil sample was compacted in accordance to BS 1377: Part 4: 1990, Clause 7.2.4.4.

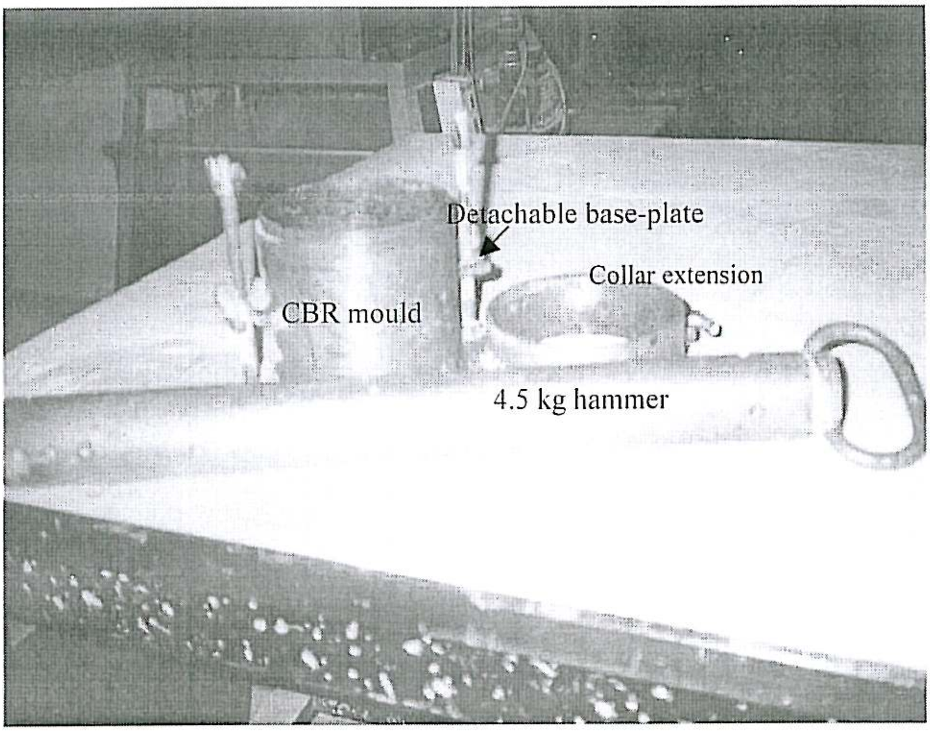


Figure 3.12: CBR mould, detachable base-plate, collar extension and 4.5 kg hammer for the preparation of CBR sample

3.4.1.2 Soaking of Specimens

The compacted CBR specimens were then soaked in the soaking tank at the normal soaking period of 4 days (BS 1377: Part 4: 1990). The level of water reached the extension collar of the mould as shown in Figure 3.13. The soaked specimens were then removed from the immersion tank and allow to drain for 15 minutes prior to the penetration test.

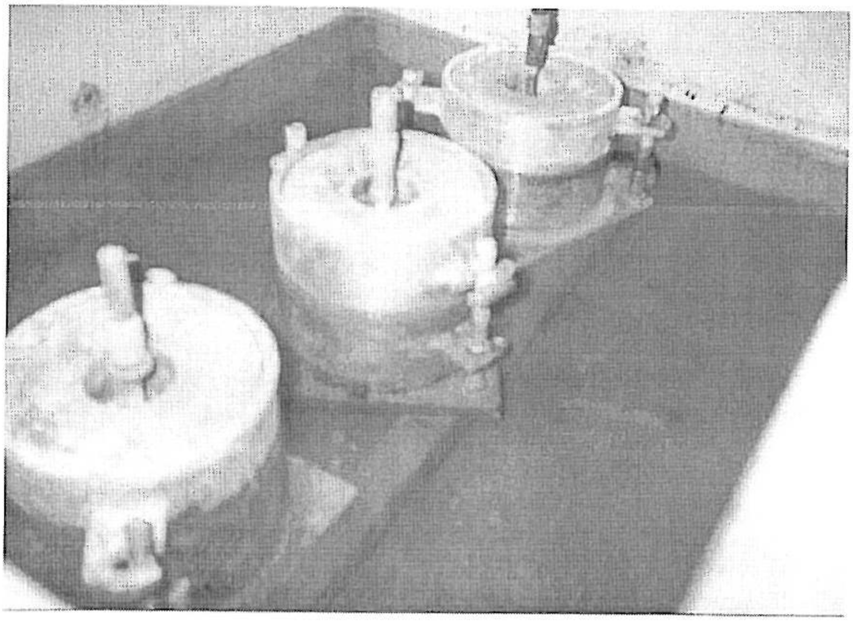


Figure 3.13: Soaking of CBR sample prior to the CBR test

3.4.1.3 Test Procedure

Force was applied at a constant rate through a plunger. The machine is capable of applying at least 45 kN at a penetration rate of the plunger of 1mm/min to within ± 0.2mm/min. A plunger is a hardened steel and have a nominal cross section area of 1935 mm^2, corresponding to a specified diameter of 49.65 ± 0.10 mm (Figure 3.14). The CBR specimen was placed centrally under the plunger until it touched the soil surface. During the test, the load applied to the specimen was measured for every 0.25

mm of the penetration. The CBR test was stopped when the penetration achieved 7.5 mm and the test procedure was repeated for the bottom layer of specimen.

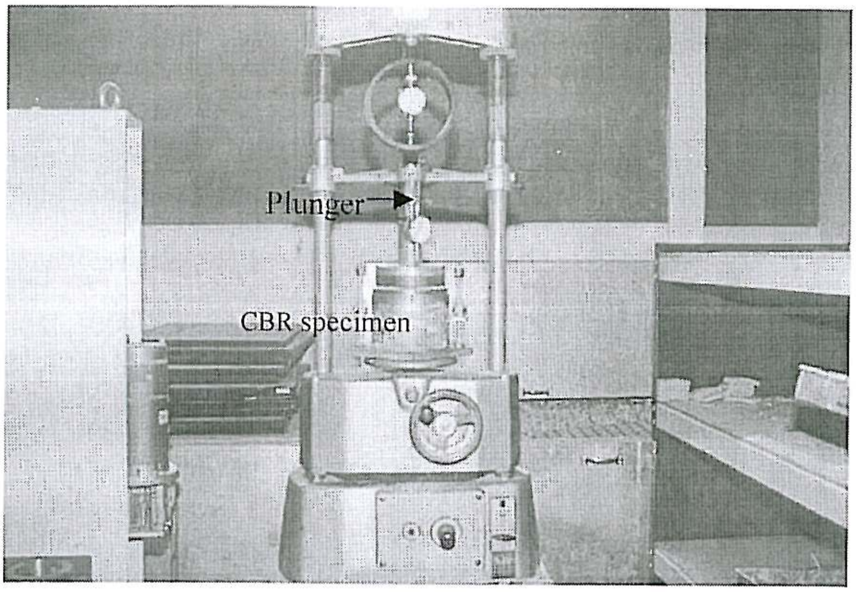


Figure 3.14: California Bearing Ratio (CBR) penetration machine

3.4.2 Unconfined Compressive Strength (UCS) and Vane Shear Test

The UCS specimens were obtained from the CBR specimens using the extruder machine and the testing was conducted according to the BS 1377: Part 7: 1990, Clause 7.2. For the Vane Shear strength test, it followed the procedure given in BS 1377: Part 7: 1990, Clause 3. Soil samples with certain moisture content tested for CBR value were also tested using Vane Shear and UCS for their undrained shear strength. From these tests a correlation between CBR and undrained shear strength is possible. The tests apparatus for the UCS and Vane Shear strength test are shown in Figure 3.15 and 3.16.

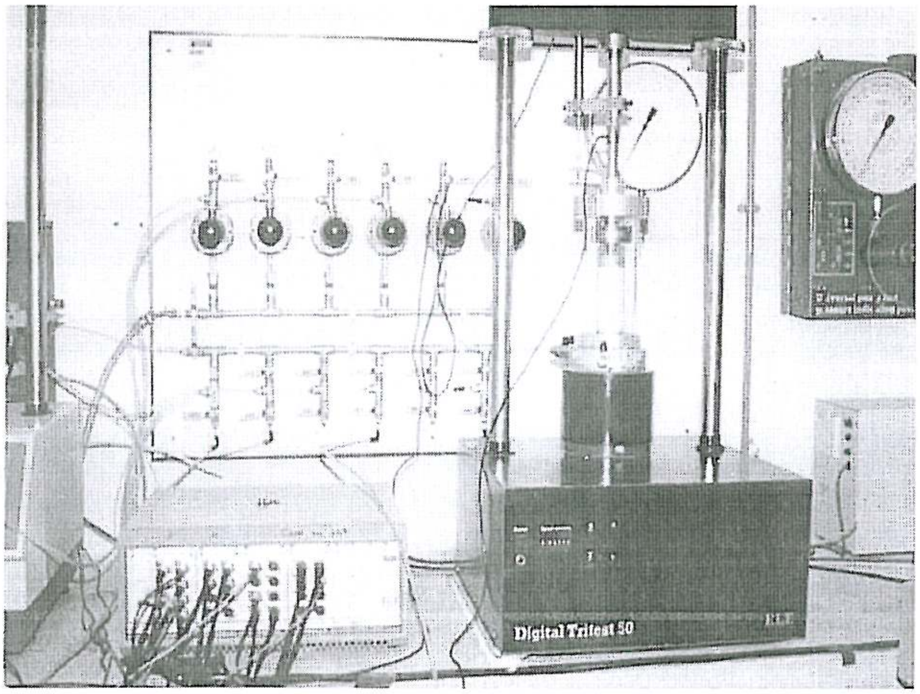


Figure 3.15: Unconfined Compressive Strength (UCS) test apparatus

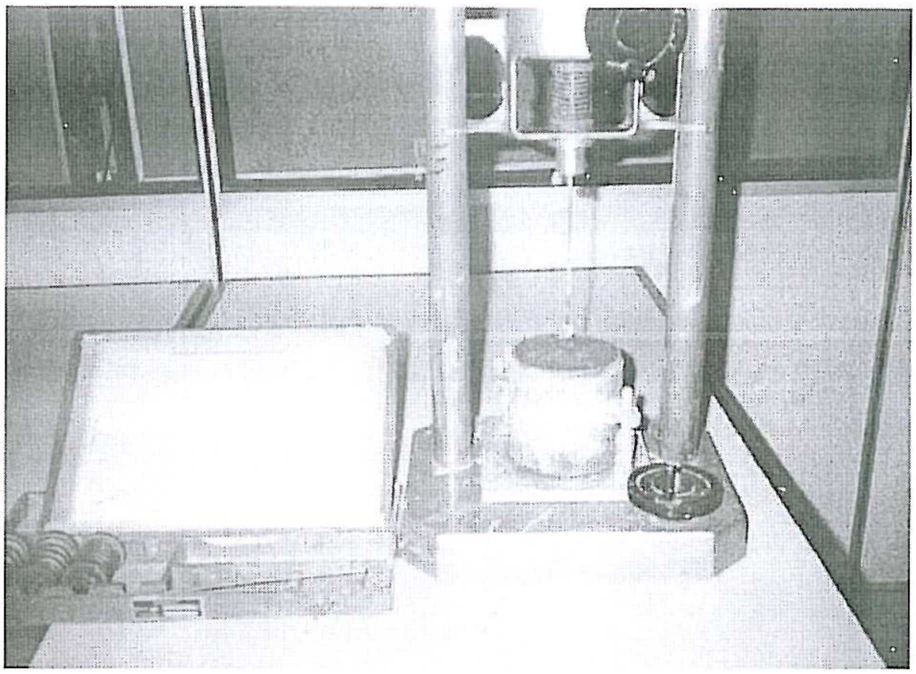


Figure 3.16: Laboratory Vane Shear test apparatus

3.5 Soil-Lime Mixture

It is important that the lime and soil are intimately mixed. In the mixing process, there must be sufficient free water in the mix to hydrate the lime and carry it into the solution to react with the clay. For the purpose of this research, the soil-lime mixture was mixed with the amount of water based on the CBR value of the soils at 2 % to represent the stabilization process of soft layer. The result from the Initial Consumption of Lime (ICL) test gives an indication of the minimum amount of lime that is needed to achieve a significant improvement in the properties of the soil. However for sub-grade stabilization to form capping, the addition of 2.5 % or greater available lime is required by the DTp Specification to ensure that maximum benefit is obtained however the actual addition rates for available lime can be established by the laboratory testing (Lime Stabilisation Manual, 1990). In this research, the percentage of lime used is 5 %.

Before the stabilization process, the clay soils were oven dried at 100 ± 10°C to ensure it is free of water. According to the Lime Stabilisation Manual (1990), during the construction at site, lime should be spread to the soil before the addition of water. After that, the soil-lime mixtures were adequately pulverized to distribute the lime through the soil. Finally, soil-lime mixtures were compacted to the specified density. The procedures were followed for the soil-lime mixtures preparation works in this research.

3.6 Mix Design and Strength Development of Stabilized Soils

The purpose in carrying out a laboratory test is to obtain the best result from the materials available, to ensure that the minimum specification requirements can be met and to establish control parameters for subsequent site work (Lime Stabilisation Manual, 1990). The soil-lime mixtures were compacted in the CBR mould according to BS 1377: Part 4: 1990, Clause 7.2.4.4. Then, the samples were soaked (section 3.4.1.2) in a soaking tank immediately for 4 days after the compaction process completed. For the strength development investigation, the CBR samples also prepared for the curing period of 7, 14, 28 and 56 days before soaking process. After the soaking period completed, the samples were test for their strength with the CBR test.

3.7 Deformation Model

3.7.1 Introduction

The objective of the model is to look the improvement of the soft layer after lime-stabilization method is conducted. The flexible pavement structures were rested on the sub-grade. Therefore, the sub-grade must have enough strength to sustain the load from the pavement structure and vehicles. If there is deformations occur at sub-grade, the pavement structures also deform with the same magnitudes (Mohamad Rehan Karim, *et.al*, 1993). Little (1995), described that the load applied on the flexible road surface is the moving wheel load and the potential deflection of the beneath layer is likely similar to wave pattern as shown in Figure 3.17. By visualizing the soil structure of lime-stabilized soil overlain soft clay and soft clay without any stabilized layer, the settlement was compared after static load has been applied. During the test, the maximum settlement was allowed is 20 mm. According to Lilley (1991), he stated that in Road Note 29 considered the bituminous surface road has failed if it developed ruts of 20 mm or more in depth.

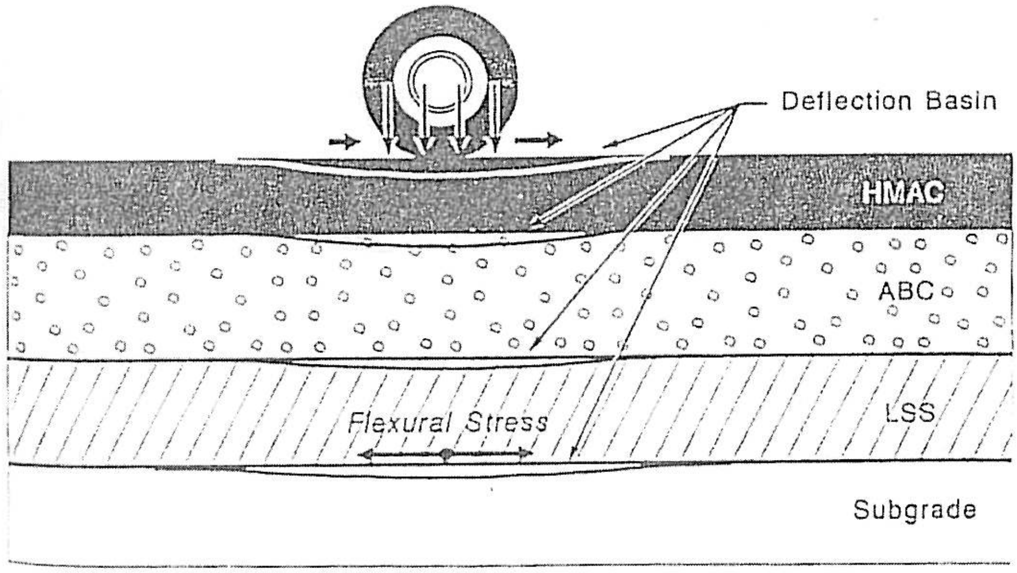


Figure 3.17: Deflection due to the wheel load in the road (after Little, 1995)

3.7.2 Consistency of Works

3.7.2.1 Calibration of the Apparatus

Before the model testing was conducted, the apparatus used especially for the measurement of load and settlement was calibrated. The proving ring (No. 9080) and Linear Vertical Displacement Transducer (LVDT) were calibrated and the results of the calibration test are shown in **APPENDIX B** and **C**.

3.7.2.2 Degree of Compaction

The degree of compaction was determined by using the compaction test method. The soil-lime mixture was prepared for the compaction test according to BS 1924: Part

2: 1990, Section Two. The result of the testing is shown in **APPENDIX D**. From the compaction test, the Maximum Dry Density (MDD) was achieved.

Lime Stabilisation Manual (1990), suggested that the soil-lime mixture should be compacted to at least 95% of the MDD and compacted on the wet side of Optimum Moisture Content (OMC). This is to have a slight excess of moisture content on the ground so that lime-clay reaction mixtures continue to gain strength. Bell (1998), mentioned that soil-lime mixture compacted at moisture content above OMC, after brief periods of curing have higher strength than those compacted with moisture content less than optimum. This is probably because the lime is more uniformly diffused and occurs in a more homogenous curing environment. Nevertheless, the strength of soils compacted at or below OMC generally can be enhanced by further addition of water after compaction.

The soil-lime mixture was prepared, placed on the tray, pulverized and left at room temperature (Figure 3.18). The moisture content value of the mixtures were determined started with an immediate samples followed by the other samples until the maximum period of 46 hours. This process is to obtain the correlation between the maturing period and the moisture content of soil-lime mixture. The result and the correlation between the moisture content value and maturing period was developed and is shown in **APPENDIX E**. From this correlation, the maturing period for soil-lime mixture to achieve 95% of MDD was determined.

3.7.2.3 Trial of the Compaction

About 10 kg of oven dried UTM Yellowish Clay soil-lime mixture was prepared. This mixture then leaved at the room temperature and allowed to maturing period as describe in section 3.1.7.2. Then the mixture was compacted in the tray using the hand-roller compactor.

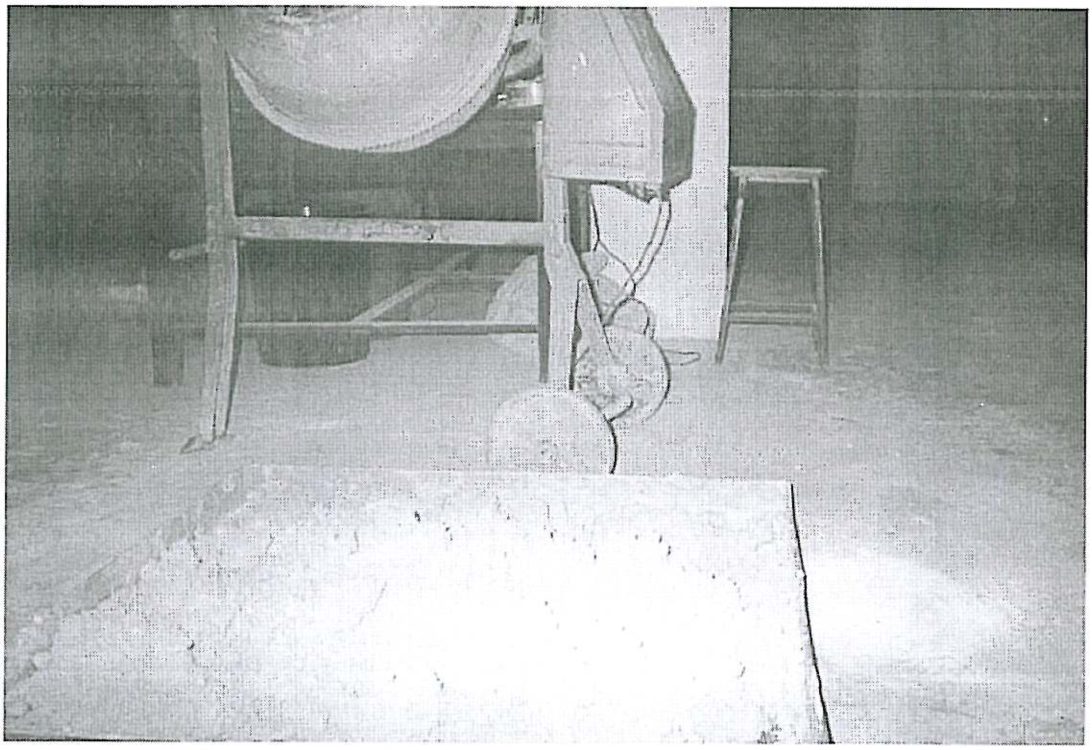


Figure 3.18: Soil's mixture and soil-lime mixture was laid on the tray after the mixing process

From the trial compaction, the number of passes of the hand-roller compactor to achieve 95% MDD for soil-lime mixture in the model was established. Basically, the concept is shown as follows:

Maximum Dry Density (MDD)	=	1.39 Mg/m3
95% of MDD	=	1.32 Mg/m3

The dry density of the compacted soil-lime mixture was determined using the Equation 3.5 given below:

$$\gamma d = \frac{\gamma b}{1 + w} \qquad (3.5)$$

where,

w is the moisture content at 95% MDD (wet side) obtained from the correlation in section 3.7.1.2.

Therefore,

$$1.32 = \frac{\gamma_b}{1 + 0.3688}$$

$$\gamma_b = 1.32x1.3688$$

$$\gamma_b = 1.81\frac{Mg}{m^3}$$

From the calculation, the bulk density of the soil-lime mixture was 1.81 Mg/m^3. The bulk density of the soil-lime mixture was obtained with the Equation 3.6 given below:

$$\gamma_b = \frac{m}{V} \tag{3.6}$$

where,

m is the mass of the soil-lime mixture

V is the volume of the tray (0.035 m^3)

$$m = \gamma_b x V$$

$$m = 1.81x0.035$$

$$m = 63.2kg$$

Soil-lime mixture then was compacted until its total weight is 63.2 kg and the thickness of the compacted soil-lime mixture was measured. As a result, compacted soil-lime mixture achieved the 95% of MDD after applied 21 passes of the roller and the thickness of the stabilized layer was achieved 30 mm. This procedure was applied to compact all samples of the soil-lime mixtures in the model.

3.7.3 Materials and Apparatus

In the preparation of samples and conducting the tests, some apparatus were used. The apparatus involved in the model testing are:

a. Soil-mixer (Figure 3.18)
b. Portable data logger (Figure 3.19)
c. Linear Vertical Displacement Transducer (LVDT) (Figure 3.20)
d. Proving Ring (No. 9080) (Figure 3.21)
e. Hand-roller compactor and compaction tray (Figure 3.22)
f. Compression plate (Figure 3.23)
g. Penetration cylinder (Figure 3.24)
h. Hydraulic pump (Figure 3.25)

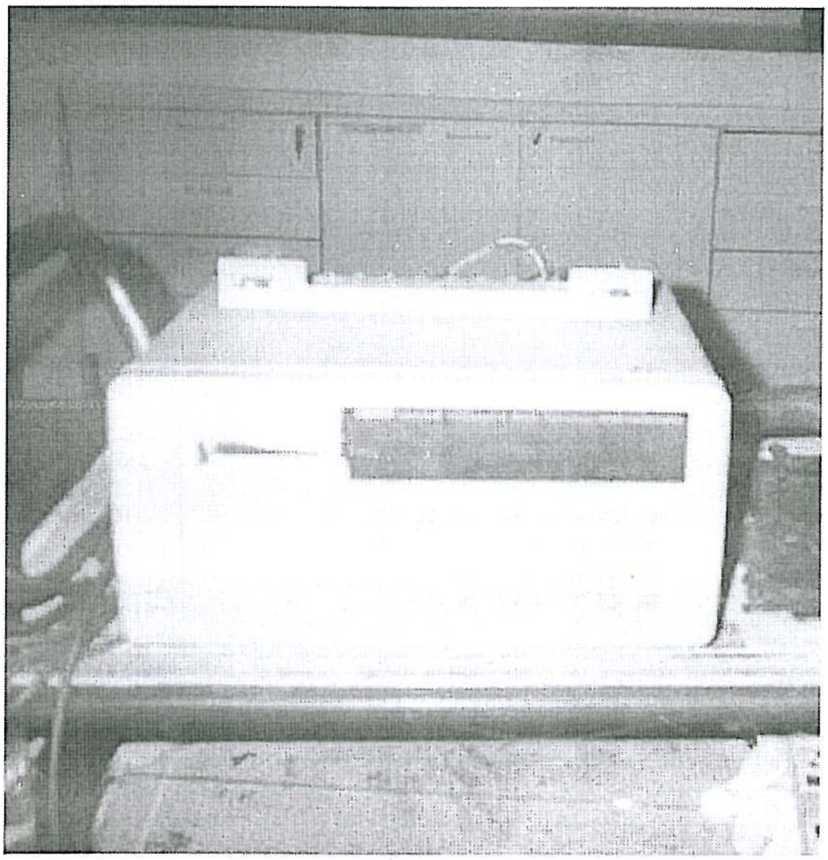


Figure 3.19: Portable data logger used to measure the model testing reading

Figure 3.20: Linear Vertical Displacement Transducer (LVDT) used to measure the settlement of the tested soil-lime mixture

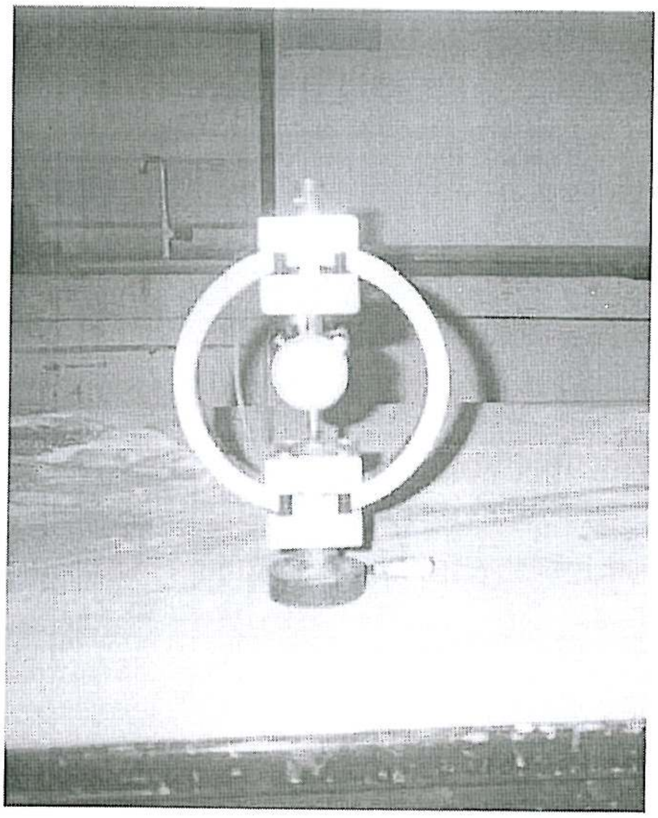


Figure 3.21: Proving ring (No. 9080) to determine the load during testing

Figure 3.22: Hand-roller for compaction and the compaction tray

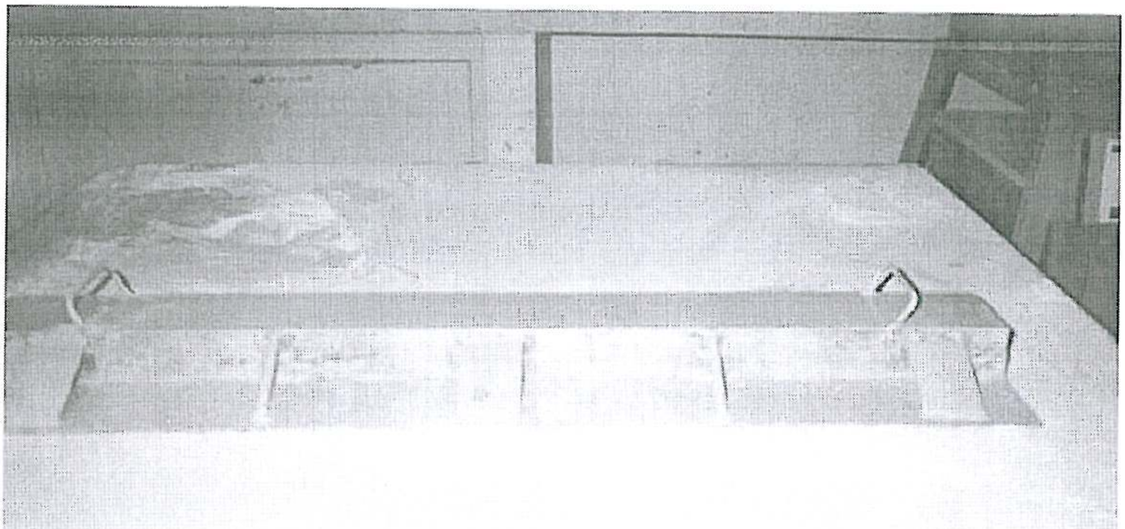


Figure 3.23: Compression plate to drain off the water from the slurry clay sample

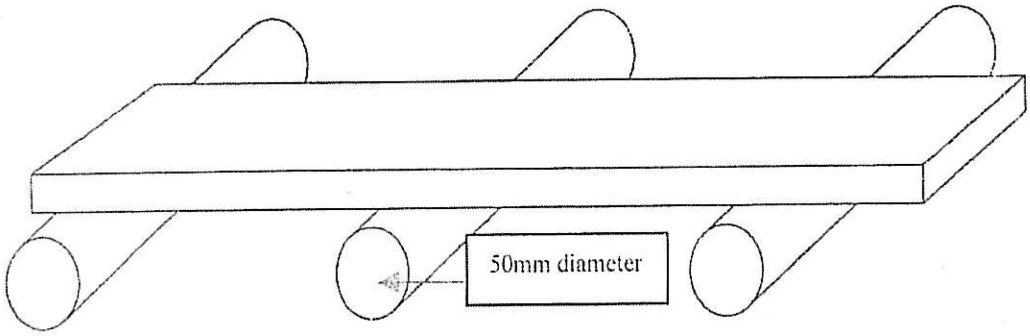


Figure 3.24: Penetration cylinder to penetrate the compacted lime stabilized soil surface

The clay soil used for the model testing was UTM Yellowish Clay. The soils was oven dried at the temperature of 100 ± 10°C. The clay soils where then stored in a tank and ready for the mixing process.

The frame of the model was fabricated from rectangular cast iron. Wall of the model was made from the acrylic product with the thickness of 25 mm. The proving ring was used for the determination of the load applied. Hydraulic jet was located et the middle of the frame, connected to the proving ring. The purpose of the hydraulic pump is to supply the pressure, which was transferred as vertical load when the proving ring tightly fixed at the top of the frame. The penetration bar then pressed the soil downward and the Linear Vertical Displacement Transducer (LVDT), which was connected to the portable data logger, measured the settlement value.

3.7.4 Schematic Diagram

The schematic diagram of the model is given in the **APPENDIX F** of this thesis.

3.7.5 Sample Preparation

In order to simulate the actual situation in the field, the natural marginal clay was prepared. The clay soil was mixed with water to prepare a slurry sample. The amount of water was based on liquid limits. The mixed material was emptied from the mixer and then placed on top of slurry surface. Pressure of 5 kPa was applied at the top of the compression plate and maintained for the 7 days. After that, the soil-lime mixture was prepared and then was compacted.

Soil-lime mixture was prepared and compacted accordance to the consistency of work in section 3.7.1. After compaction, soil-lime mixture was cured at the different curing period prior to test. There were 4 samples of compacted soil-lime mixtures and each sample was cured at room temperature from 0, 7, 14 and 28 days. The tests were also conducted to an unstabilized sample as a control sample.

3.7.6 Experimental Set-Up

When the unstabilized and stabilized samples were prepared, the penetration test was applied. The penetration cylinder was centrally placed on the surface of the sample. A hydraulic pump was placed on the penetration cylinder and connected with proving ring. The linear vertical displacement transducers (LVDT) was placed at the penetration cylinder and connected to portable data logger to measure the settlement. The dial gauge of proving ring was reset to zero and the test was ready to conduct. The experimental setup for the model testing is shown in Figure 3.25.

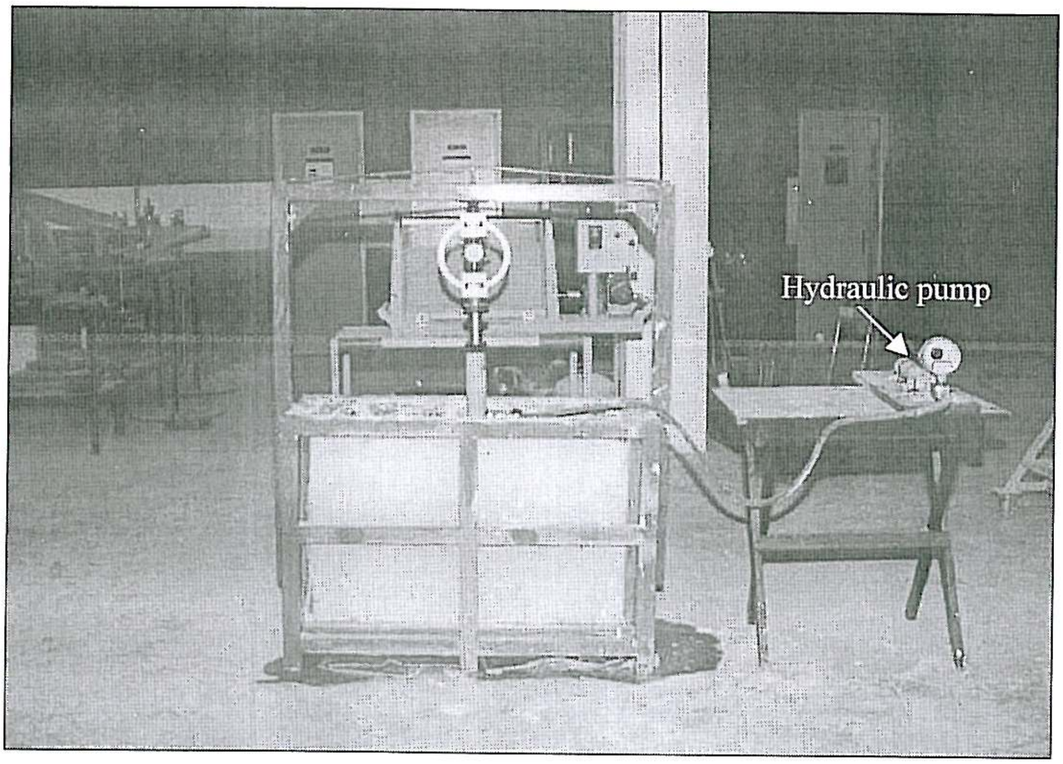


Figure 3.25: Experimental ~~setup~~ for visual inspection model

set-up

3.7.7 Test Procedure

When all of the apparatus was ready, the test was started with pushing the hydraulic pump. The settlement was measured for every 100 kPa pressure from the hydraulic pump. The loading was stopped when the settlement of the sample achieved 20 mm of depth.

3.8 Bearing Capacity Model

The bearing capacity of the stabilized soil and soft clay structure was also investigated. In order to establish the dimension of the model, two parameters were involved. First parameter is the effective contact width of the vehicle's tyre and the second parameter is the total thickness of the road structure (excluding sub-grade). In this research, the maximum and minimum width of tire for commercial vehicles used for the model analysis is 275 mm and 135 mm respectively. This value is based on the common width of commercial vehicles tyre used in Malaysia. For the consideration of the road thickness, it was based on the thickness obtained from the Overseas Road Note 31 (1993). This standard gives a guide to the structural design of bitumen-surface roads in tropical and sub-tropical countries. As a reference, Chart No. 5 was used because it proposed the use of granular material for the road base and structural surface, which are commonly roads structures constructed in Malaysia. From the chart, road thickness was obtained based on the sub-grade strength from the CBR (%) value and the classes of the traffic from the equivalent standard axle (ESA x 10^6). The critical condition of the sub-grade with CBR value of 2 % (S1) was considered. At this stage, the maximum and minimum road thickness, H_R is 525 mm and 650 mm respectively.

Other aspect, which has been focused on this research, is the consideration of the road structure for the low traffic intensity. According to Road Note 31 (1993), for the low traffic intensity, it only used the surface dressing as a pavement structure. Based on the Chart No. 1, the maximum and minimum thickness of surface dressing is 575 mm and 350 mm for the sub-grade CBR value of 2 %. As the result, the maximum and minimum thickness of the road is 650 mm and 350 mm was used in this research.

The contact area above the sub-grade surface was measured. Kenny and Andrawes (1997) mentioned that Terzaghi and Peck assumed that the load spreads through an angle corresponding to two vertical units for every horizontal unit of distance (i.e., a load spreading angle, α where tan $\alpha = 0.5$) as shown in Figure 3.26. Using the road thickness of 650 mm and 350 mm, the maximum and minimum width of the contact area was calculated by the Equation 3.7. As the result, the maximum and minimum width of the contact area (B_{Max} and B_{Min}) is 925 mm and 485 mm. For the model construction, these values were reduced with the scaling factor of 10. After considered the tolerance of the load-spreading angle, the actual size of the model frame for bearing capacity model testing is shown in Figure 3.27.

$$B_{eff}\ (\mathrm{B_{Max}\ or\ B_{Min}}) = B + H_R \quad \text{for load spreading angle } 2:1 \tag{3.7}$$

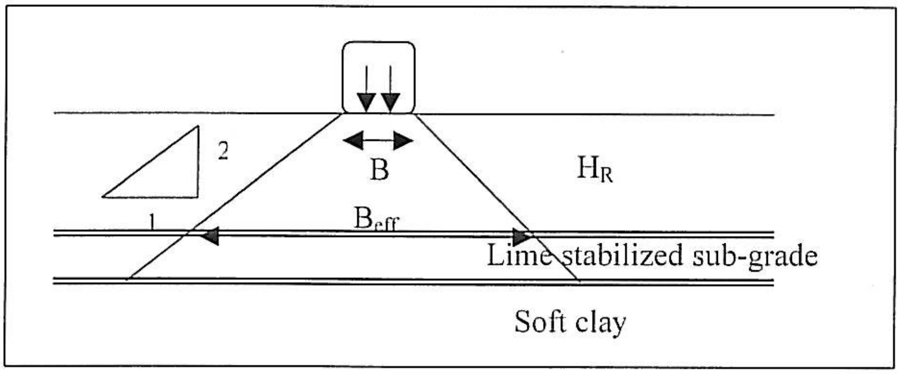


Figure 3.26: Terminology of the measurement the effective contact area for the lime stabilized sub-grade as a capping layer for road construction

where,

B - The effective width of the vehicles tire

H_R - Road thickness

B_{eff} - Maximum or minimum width of the contact area of the stress from surface

3.8.1 Materials and Apparatus

The materials and apparatus required for the test are as follows:

a. Bearing capacity model frame – Fabricated from angle iron as a frame and acrylic product ¼" thick for its wall. At the bottom, holes were made and this perforated bottom can allow drainage (Figure 3.28).

b. Model testing frame – includes a platform to put the model frame, a motor and gear for applying the penetration load (Figure 3.29).

c. Load cell – has maximum loading capacity of 3 kN (Figure 3.30).

d. Linear Vertical Displacement Transducer (LVDT) – with displacement 100 mm for measurement (Figure 3.20).

e. Portable data logger – to collect all of raw data during the test (Figure 3.19).

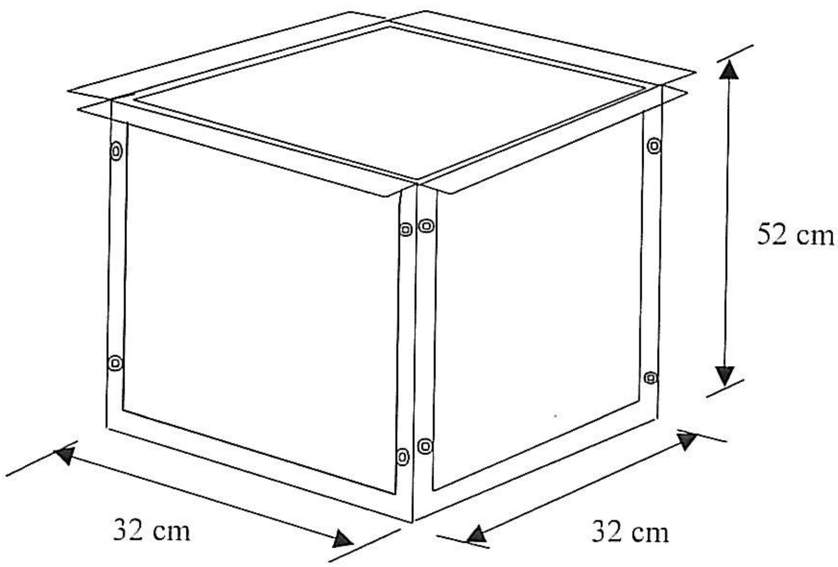


Figure 3.27: Full dimension of model frame used for the bearing capacity model

Figure 3.28: Bearing capacity model frame – soft clay and stabilized soil was compacted in this frame

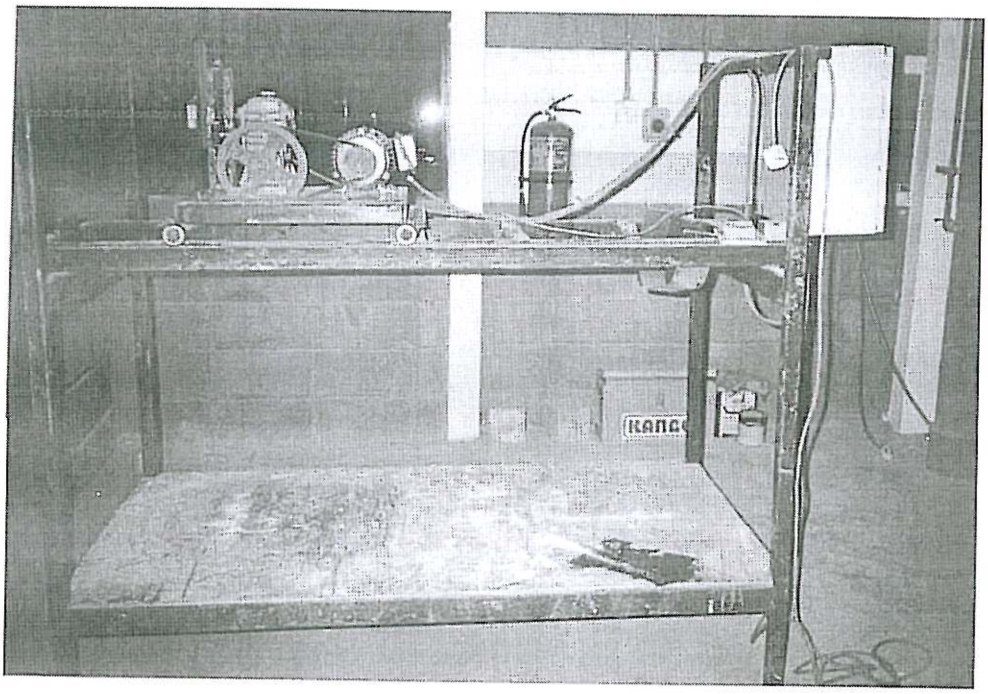


Figure 3.29: Frame for the bearing capacity model testing

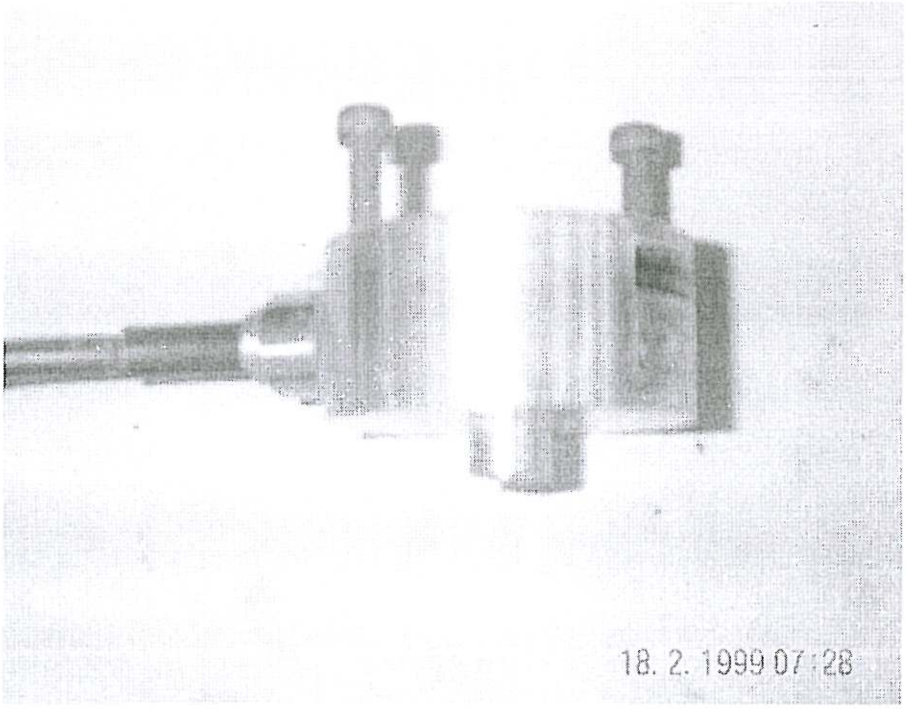


Figure 3.30: Load cell to measured the load applied during the bearing capacity testing

3.8.2 Schematic Diagram

The schematic diagram for the whole experimental apparatus was illustrated in the **APPENDIX G**.

3.8.3 Sample Preparation

3.8.3.1 Lime Stabilized Thickness

According to Sherwood (1993), for the clay soil, with CBR value of less than

2 %, the lime stabilized capping layer, which need to be constructed is 600 mm. Using the scaling factor of 10, the thickness of the lime stabilized capping layer was reduced to 60 mm. This research also investigated the bearing capacity with the different thickness of stabilized soil underlain weaker soil. The other thickness of the stabilized soil was prepared for 40 mm and 80 mm. Figure 3.28 showed the samples prepared prior test with unstabilized sample and stabilized layer with the different thickness from 40 mm, 60 mm and 80 mm.

3.8.3.2 Slurry Sample

The slurry sample was prepared by mixing dried UTM Yellowish Clay with water. The water content was equivalent to Liquid Limit (LL) of the clay soil. The clay soil was mixed with water in the mixer and the slurry sample was placed in the model frame. The sample was then surcharged by applying 5 kPa pressure at the top of its surface for 7 days as shown in Figure 3.31. The water was drained out through the holes at the bottom of the model. The holes were fitted with fine copper mesh, to allow only water to drain through. This procedure was similar to the preparation of the slurry sample for the visual model in the section 3.7.4. Then the unstabilized soil was ready to underlain by the stabilized layer as shown in Figure 3.32.

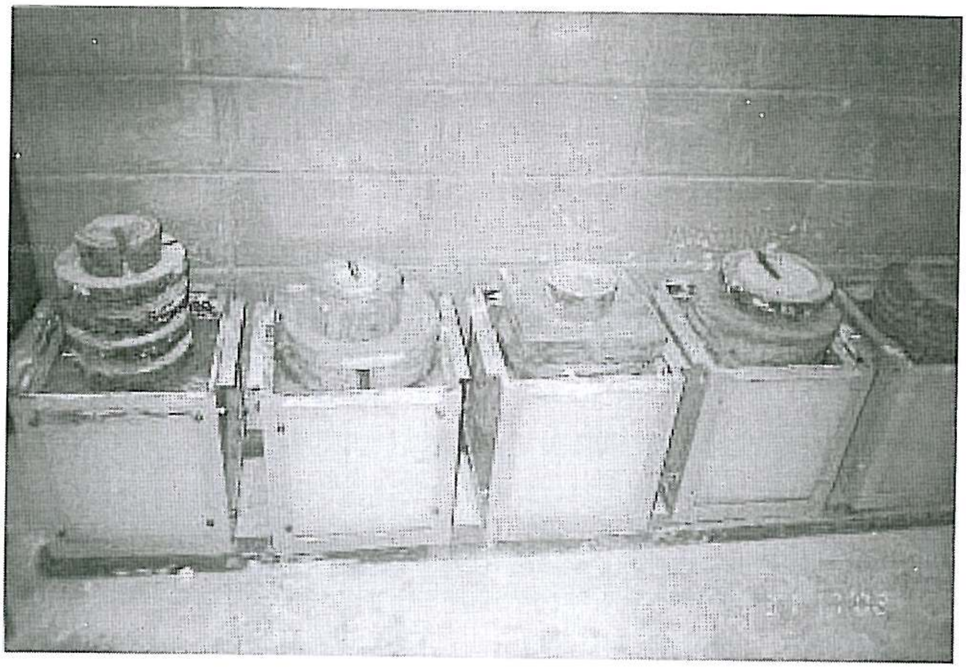


Figure 3.31: Slurry sample was surcharged at 5 kPa for 7 days

Figure 3.32: Slurry sample was ready for the stabilization process

3.8.3.3 Soil-lime Mixture

After the thickness of the stabilized layer was determined, the preparation of the soil-lime mixture was conducted. The work stared with the mixing of the oven dried UTM Yellowish Clay with OMC and 5% of lime based on mass of dry soil. Then the soil-lime mixture was compacted according to 95% of MDD on top of the soft clay layer until the required thickness level was achieved. The sample was than cured and ready for test. Four compacted soil-lime mixture with different curing periods of 0, 7,14 and 28 days were prepared.

3.8.3.4 Experimental Set-Up

The bearing capacity model frame with the compacted soil-lime mixture was placed on the platform under the penetration gear. Penetration bar was connected to the gear and the load cell at the end of the bar. Penetration gear was push downward the load cell touches the rectangle bar. The Linear Vertical Displacement Transducer (LVDT) was placed at the top of the rectangle bar to measure the settlement. Load cell and LVDT were then connected to the portable data logger and all the readings were then reset. Switched on the induction motor and penetration gear slowly pushed the penetration cylinder downward until it penetrated the soil (Figure 3.33).

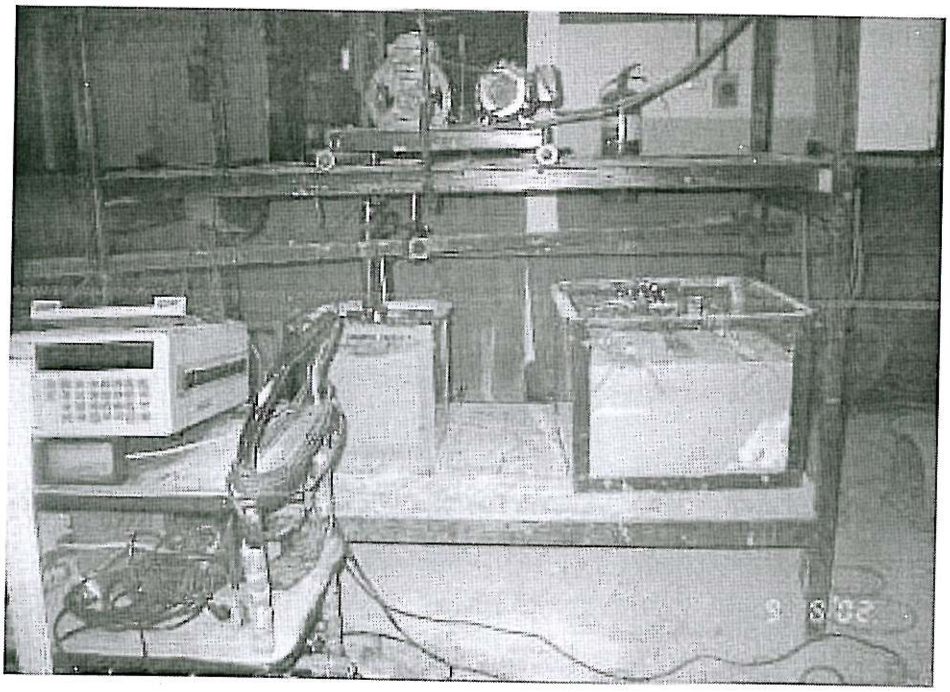


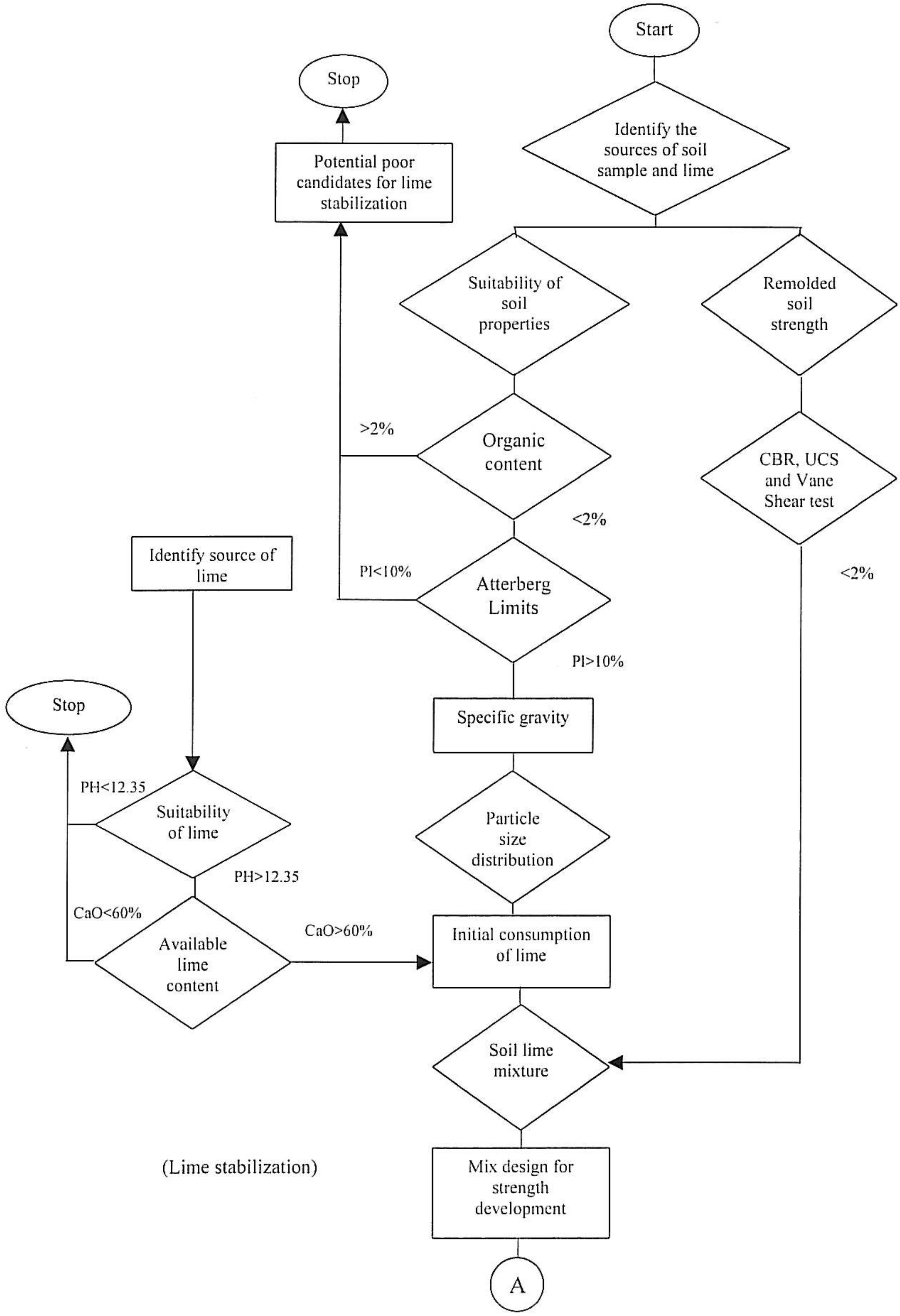
Figure 3.33: Experimental set up for the bearing capacity model

3.8.3.5 Test Procedure

When all the apparatus was ready for the test, the induction motor at a rate 0.5 rpm was switched on. This speed was chooses based on the interval reading of the data logger at every 5 seconds and this allowed the data measured to be distributed evenly. The test continuously conducted until 20 mm penetration of the soil was achieved. The test was then repeated for all compacted soil-lime mixture samples with the same procedure.

3.9 Summary

This chapter, covers the research methodology, present the soil material used, the quality of hydrated lime, CBR samples preparation and curing condition, strength development of the stabilized soils and physical models testing to look the stiffness, bearing capacity and performance of the lime stabilized sub-grade as a capping layer for road construction. Figure 3.34 presents a flowchart that describes the conceptual research methodology.



(Lime stabilization)

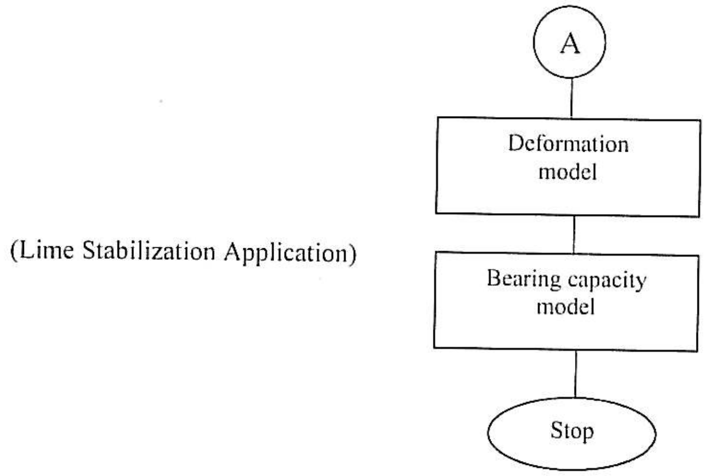


Figure 3.34: Conceptual research methodology

CHAPTER IV

RESULT AND DISCUSSION

4.1 Introduction

Results from the tested soils, lime, soil lime mixtures and physical models will be discussed in this chapter. From the result, it will determine the suitability of soils and lime before the stabilization process. Beside that, the mix design value of soil-lime mixture for the stabilization was obtained. Three types of clay soils with the PI value ranges from 20 to 35 % were stabilized with lime to establish the strength development. As to provide an understanding on the road construction technique using lime stabilization method for sub-grade of cohesive soils as capping layer, the physical model was developed. For the model testing, the UTM Yellowish Clay soil was used. From the strength development result, this type of soil shows the consistent development of strength with the curing period. The soft layer for the model testing was prepared based on water from the Liquid Limit (LL) value. The oven-dried clay was mixed with the water and then was surcharged for 5 kPa in 7 days. From the model, it showed the increasing of bearing capacity of the lime stabilized capping layer overlay soft layer with the curing period and the thickness of the stabilized layer.

4.2 Tests for Soil Properties

In order to identify suitable soil for lime stabilization, soil tests need to be carried out. The results obtained are based on tests which has been discussed in Chapter 3 as follows:

i. Particle density
ii. Particle size distribution
iii. Atterberg Limits
iv. Organic content
v. Soil Classification

4.2.1 Particle Density

In the BS 1377: Part 2: 1990, Clause 8.1 uses the term particle density instead of specific gravity, which was used in the previous editions of this standard. To comply with the current usage in other standards, it is denoted by the symbol 'ρ_s'. In this standard, particle density is quoted in Mg/m^3, which is numerically equal to the specific gravity. The method of the test used is small pyknometer method. This method is suitable for soil containing particles finer than 2 mm. The particle density of the soil was calculated from Equation 4.1.

$$\rho_s = \frac{m_2 - m_1}{(m_4 - m_1) - (m_3 - m_2)} \tag{4.1}$$

where,

m_1 is the mass of density bottle (in g)
m_2 is the mass of bottle and dry soil (in g)
m_3 is the mass of bottle, soil and water (in g)
m_4 is the mass of bottle when full of water only (in g)

The values of the particles density were expressed to the nearest 0.01 Mg/m^3 and the result of the soil particles density are given in Table 4.1. Working example for the calculation is given in **Appendix H**.

Table 4.1: Specific gravity for the unstabilized soils

Soils	Specific gravity (SG)
Tg. Pelepas marine clay	2.67
UTM yellowish clay	2.60
Kulai pinkish white clay	2.67

4.2.2 Particle Size Distribution

Soils are primarily on the basis of the particle size distribution. Particle size has control over many aspects of the engineering behavior of soil. Sand and gravel are cohesionless particles that posses no inter-particle bonds. Clay and silts are cohesive, which their particles are very small and posses enormous surface are and highly charged in nature, therefore, the clay can readily absorb polar liquids like water and cation available in the surrounding sources.

Table 4.2: Percentage of particle size for various types of clay soils

Soil description	Tg. Pelepas Marine Clay (%)	UTM Yellowish Clay (%)	Kulai pinkish White Clay (%)
Gravel (60.0 – 2.0 mm)	0.0	0.0	0.0
Sand (2.0 – 0.06 mm)	22	15	27
Silt (0.06 – 0.002 mm)	38	49	39
Clay (< 0.002 mm)	40	36	34

BS 1377: Part 2: 1990 defined the particle size distribution as the percentage of various grain sizes present in a material as determined by sieving and sedimentation. The particle size ranges adopted in civil engineering works are indicated in the first column of Table 4.2. From Table 4.2, it is also indicated that the entire soil specimens are free from gravel. Sand particle for Tg. Pelepas Marine Clay is 22%, UTM yellowish clay is 15% and Kulai pinkish white clay is 27%. For the clay particles, Tg. Pelepas Marine Clay consists of 40%, UTM Yellowish Clay is 36% and Kulai Pinkish White Clay is 34% respectively. The clay fraction composed of particles of sizes finer than 0.002 mm or 2m. In order to get the value of clay fraction for the samples, particle size distribution curve of unstabilized soils from all site locations was plotted as presented in Figure 4.1. From the curve, all soils tested contain clay fraction of more than 20%. Results of the wet sieving and hydrometer tests are attached in **Appendix I**.

Lime has little effect in soil with little or no clay content and it is most effective to the stabilization of heavy clay soils (Ingles and Metcalf, 1972). All clay minerals react with lime; however, the rate of reaction may vary, and some clay-bearing soils require more lime than others. Generally, lime also react with most silts and loams and many granular soils, as long as these materials contain at least 10% of clay content, since this is the key to the chemical reaction which provide immediate and longer term improvement in soil properties (McDowell, 1972).

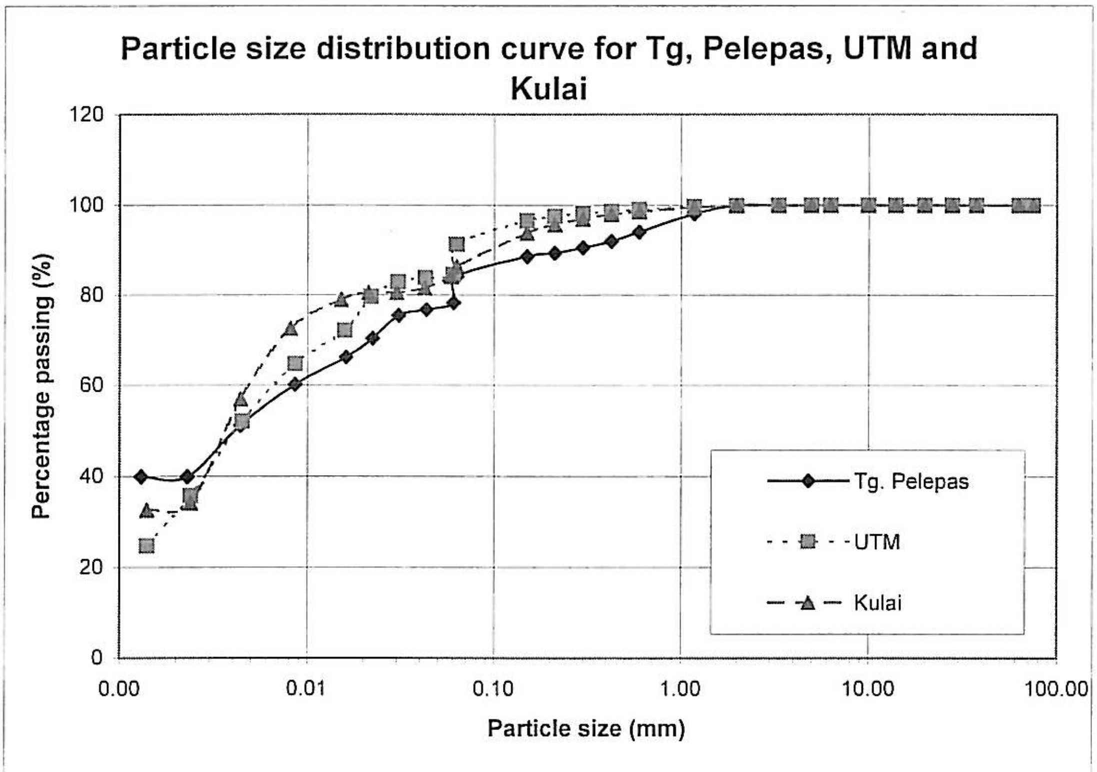


Figure 4.1: Particle size curves of unstabilized soils

4.2.3 Soil Classification

Soil classification system is a basic language by geotechnical engineers to name a soil. It provides a logical basis for accumulating data pertaining to physical properties of soils and their correlation with certain soil types. Soil classification is usually accomplished by means of the plasticity chart; or is sometimes referred to as the A-Line chart. It is a graphical plot of the liquid limit (LL) as ordinate and against plasticity index (PI) as abscissa. The charts in accordance with BS 5930: 1981 and Unified Soil Classification System (USCS) were used to classify the soil. These charts provide most useful way of identifying and classifying the fine-grained soils. Table 4.3 gives a summary of the soil classification in accordance with the stated standards. All the relevant values of LL and PI can be obtained in Table 4.4.

Table 4.3: Soil classification using BS 5930 and USCS for unstabilized soils

Specification	Tg.Pelepas	UTM	Kulai
BSCS	CH	CH	MH
USCS	CH	CH	MH

In accordance to BS 5930: 1981, Tg. Pelepas marine clay and UTM yellowish clay can be classified as high plasticity clay (CH). Only Kulai pinkish white clay is classified as high plasticity silt (MH). The other results based on the USCS, all the collected soil samples with the exception of Kulai pinkish white clay are classified as inorganic clays of high plasticity or as fat clay (CH). Kulai pinkish white clay has a group with the symbol MH, which is described as inorganic silts, micaceous or diatomaceous, fine sand or silts and elastic silts. It showed that, most of the soil samples for this research have a group of the clayey soils.

4.2.4 Atterberg Limits

The specification for lime-stabilized materials has been based on experience and research extending over many years. Many countries have their own national specifications, which are based on their climatic conditions. In UK, the Department of Transport (DT_p) detailed in the DT_p Specification Part 2 (Brown Book) materials suitable for lime stabilization in the road construction.

Currently, no local specification has been established in Malaysia with regards to environmental and climatic condition. Therefore, the criteria of the lower acceptable limits of plasticity greater than 10, which was specified as class 7E Materials in the DT_p Specification Part 2 was selected as a guideline. In this case, soil materials, which have plasticity index ranging from 20 to 50, were collected and the actual plasticity index are given in Table 4.4. Details of Atterberg Limit test result are attached in **Appendix J**.

The ratio of Plasticity Index (PI) to silt or clay content of a soil is useful way of correlating these two basic properties to assess the soil's suitability for lime stabilization. Lime Stabilisation Manual (1990) mentioned that the correlation between these two values is given in the Equation 4.2. Reactivity Index (RI) or clay activity (A_c) should be greater than 0.5 for lime modification affects to be beneficial and greater than 0.75 for full stabilization, subject to a minimum PI of 18.

$$\text{Reactivity Index (RI)} = \frac{\text{Plasticity Index}}{\text{\% Clay Fraction}} \tag{4.2}$$

Based on RI or clay activity (A_c), clays were classified into four groups as inactive clay, normal clay, active clay and highly active clay as shown in Table 4.5. Based on the Equation 4.2, the RI value for Tg. Pelepas marine clay is 0.8085 and UTM yellowish clay is 0.8206. Kulai pinkish white clay showed the lowest value of A_c, 0.7194 and the result are tabulated in Table 4.4. Basically, all clay soils used for this research were described as normal clay and RI value was greater than 0.75. Therefore it is suitable for the purpose of the stabilization process accepts Kulai pinkish white clay, which was described as inactive clay and RI value was less than 0.75. For the silt predominates, the soil will permanently modify by lime to reduce moisture susceptibility, but may not develop much pozzolanic strength subsequently.

Table 4.4: Physical properties of cohesive soils used

Soil properties	Soil description		
	Tg. Pelepas	**UTM**	**Kulai**
Natural moisture content (%)	121	65	45
Liquid limit (%)	56.00	57.65	50.12
Plastic limit (%)	23.66	28.11	28.54
Plasticity index (PI)	32.34	29.54	21.58
Clay fraction (%)	40	36	34
Clay activity (A_c)	0.8085	0.8206	0.6347

Table 4.5: Clay activity definition (after Kok, Kai Chern, 2000)

Description	Clay activity, A_c
Inactive	<0.75
Normal	0.75 – 1.25
Active	1.25 – 2.00
Highly active	> 2.00

4.2.5 Organic Content

The upper limits of organic matter content as recommended by the DTp, UK (1986), should not be greater than 2 %. Table 4.6 gives a summary of the organic content of various soils with Tg. Pelepas Marine clay giving the highest values of organic content, which is greater than 2 %. The presence of organic matter can be detrimental to lime stabilization reactions, but it is the type rather than the total amount of organic matter, which is the critical factor. Measurement of the total organic content of soil is therefore a poor guide for determining the suitability of a soil for stabilization. A test for detecting active organic matter is given in BS 1924 (1975 Edition).

Alternatively, the Initial Consumption of Lime Test (BS 1924: Part 2, Clause 5.4) will give a good indication of what can be achieved with organic soils. Generally, increasing the quantity of lime added o the soil would help to overcome the effect of organics (Lime Stabilisation Manual, 1990).

The value of organic matter content was obtained from the Loss on Ignition (LOI) test. LOI is the most useful index for evaluating the degree of weathering, because of its precision and feasibility for testing. The value for LOI is related closely to the amount of crystallized water and organic matter content. Adsorbed and chemically combined water cannot be removed by oven drying at 110°C and therefore considered to be part of the solid soil grain. Adsorbed water is held on the surface of the particle by powerful forces of electrical attraction and virtually in a solid state (Kok K.C., 200). Base on this statement, the crystallized water content also contributed to the value of organic matter content. According to the soil properties of the samples for this research, Tg. Pelepas marine clay showed the highest value of organic natural moisture content, followed by UTM yellowish clay and Kulai pinkish white clay. This result may affect the result of the organic matter content. The data related for this test was attached in **Appendix K** of this thesis.

Table 4.6: Organic content of unstabilized soils

Soil description	Organic content
Tg. Pelepas marine clay	5.80
UTM yellowish clay	1.50
Kulai pinkish white clay	0.47

4.3 Test for Hydrated Lime

4.3.1 Suitability of Lime

BS 1924: Part 2: 1990; Clause 5.4.6 suggested the average pH of hydrated lime should be within the range of pH value of 12.35 to 12.45 at the corrected temperature of 25°C. Lime is a strong alkali with a low solubility in water giving a maximum pH of 12.4. This pH value is required to maintain reaction between the lime and any reactive components in the material to be stabilized (Sherwood, 1993). Table 4.7 shows result of the tests conducted. Based on the result, the quality of lime meets the requirement of the Initial Consumption of Lime (ICL) test, which will be discussed in section 4.3.3. The related data for this test is attached in **Appendix L.**

Table 4.7: Determination of the suitability of lime by pH test

pH test	T°C	pH_T	pH_{25}
1	26.7	12.35	12.40
2		12.40	12.39
Average			12.40

4.3.2 Available Lime Content (ALC)

The available lime content of a calcareous material is the calcium present as oxide or hydroxide. These materials were involved in the reaction during the stabilization process. The stabilizing effect depends, ultimately, on chemical attack by the lime on the clay minerals in the soil to form cementitious compounds (Ingles and Metcalf, 1972). For the purpose of this research, the lime used was tested its available lime content before the stabilization process to assure it will react effectively with the cohesive soil and contribute to the strength development. Results of ALC tests are given in Table 4.8. The average ALC for the lime used is 67.30 %, in terms of CaO content. Percentage available lime as $Ca(OH)_2$ is 88.92 %. Depending on the source and type of product, available lime contents can vary from less than 50 % to greater than 90 % (Lime Stabilization Manual, 1990). From the result, the hydrated lime was suitable to be used in the stabilization process for the research. The result of the test is attached in **Appendix M**.

Table 4.8: Available Lime Content (ALC) test results for the Hydrated lime ($Ca(OH)_2$) used in the research

Test No.	Volume of hydrochloric acid (1M) used	% of CaO	% of $Ca(OH)_2$
1	24.8	69.54	91.88
2	23.2	65.05	85.96
Average		67.30	88.92

4.3.3 Initial Consumption of Lime (ICL)

The result from this test gives an indication of the minimum amount of lime needs for the improvement of the soil properties at early stage. However for the stabilization process or log-term reaction, the lime should be greater than the ICL value. It does not dispense with the need to carry out strength determination, as it does not establish whether the soil will react with lime to produce a substantial strength increase. Research has also suggested that lime percentage obtained from the test does not produce maximum cured compressive strength for tropical and sub-tropical soils (Sherwood, 1993). Table 4.9 summarized the Initial Consumption of Lime (ICL) test result on various types of soil at the corrected temperature 25°C (see Equation 3.1) and each ICL test result is attached in **Appendix N**.

Table 4.9: Initial Consumption of Lime (ICL) test result showing the pH values of soils at temperature of 25°C

Soil description	pH value									
	Proportion of lime (%)									
	0.0	0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0	6.0
Tg. Pelepas	3.70	11.55	11.80	12.32	12.36	12.41	12.40	12.45	12.46	12.48
UTM	3.61	11.81	12.26	12.42	12.33	12.46	12.41	12.44	12.45	12.47
Kulai	8.47	10.42	11.77	12.24	12.27	12.44	12.45	12.48	12.27	12.53

Figure 4.2 shows the curve of ICL test for the various types of soils. From the curve, the amount of lime to achieve pH value of 12.40 is about 3 % for all types of soils. Kok, Kai Chern (2000), mentioned that the correlation between ICL and clay activity (A_c) is given in Equation 4.3. From the equation, the percentage of lime for ICL test is 2.92% for Tg. Pelepas, 2.9% for UTM and 3.11% for Kulai clay. The value obtained was closed the value achieved from the curve.

$$\text{ICL}\ (\%) = -1.13A_c + 3.83 \tag{4.3}$$

From Lime Stabilisation Manual (1990), the DTp Specification requires a minimum of 2.5 % available lime. Based on the result from the curve and calculation, it is possible to take the value of 3 % of lime for the modification of the clay soils used. Table 4.10 sets out suggested lime contents for a range of soil types. From the table, it suggested 1 – 3 per cent of lime to the modification and 2 – 8 per cent of lime to the stabilization process for the clay soils. Based on this statement, the amount of lime for the stabilization stage was sets at 2 %. Soil-lime mix design, therefore, consists of 5 % amounts of lime during the test of strength development and model.

Table 4.10: Suggested lime[#] contents (after Ingles and Metcalf, 1972)

Soil Type	Content for Modification	Content for Stabilization
Fine crushed rock *	2 – 4 per cent	Not recommended
Well graded clay gravels	1 – 3 per cent	~ 3 per cent
Sands **	Not recommended	Not recommended
Sandy clay	Not recommended	~ 5 per cent
Silty clay	1 – 3 per cent	2 – 4 per cent
Heavy clay	1 – 3 per cent	3 – 8 per cent
Very heavy clay	1 – 3 per cent	3 – 8 per cent

Hydrated lime $Ca(OH)_2$

* Lime only effective if fines are plastic

** Lime used in bitumen stabilization for promoting adhesion: Quicklime in loessal materials

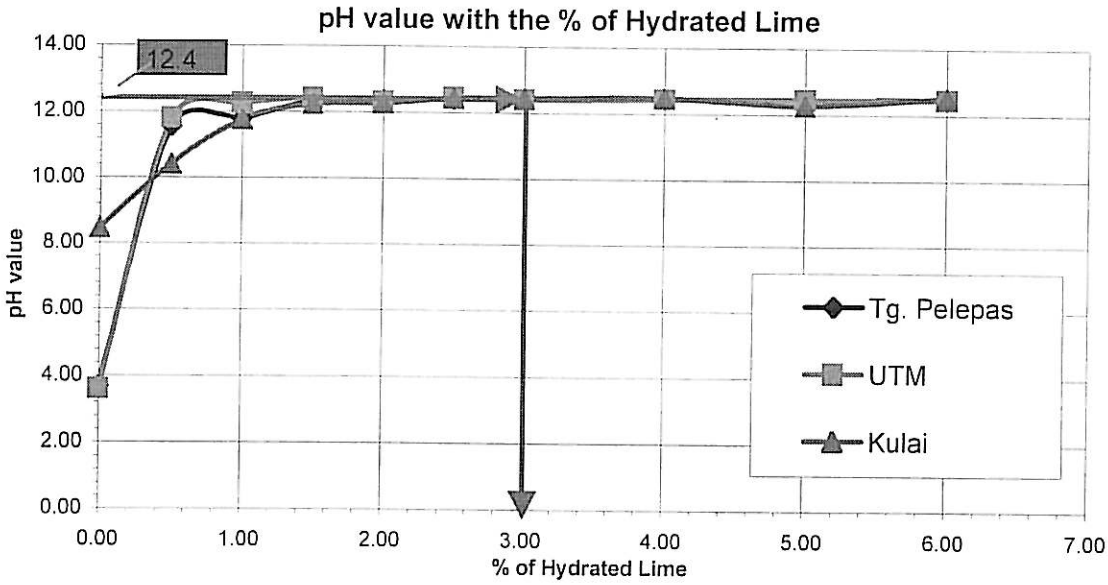


Figure 4.2: Initial Consumption of Lime (ICL) curve for unstabilized soils

4.4 Remoulded Soil Strength

Cohesive soils may be classified according to their plasticity properties. Silts have low plasticity indices, which means that they quickly become difficult to handle once the moisture content exceed the plastic limit (Sherwood, 1993). From the test, it was clearly observed that it was difficult to handle the clay soil when the moisture content increased. The strength of the clay soil was reduced with the increasing of the moisture content and this situation will contribute a problem for the engineering purposes especially for the foundation work.

4.4.1 California Bearing Ratio (CBR)

CBR test was conducted for each sample and available data for the test is given in **Appendix O**. Table 4.11 gives the summarized result of CBR value and the corresponding moisture content for each specimen. The result of CBR and moisture content was then plotted as shown in Figure 4.3, 4.4 and 4.5. From the curves, it showed the correlation between CBR value and moisture content and that curves were used to get the water content for the soil-lime mixture preparation. It was exactly defined that when the moisture content of the clay soil increase, the strength of the clay soil will reduced until no more strength achieved. At this situation, the soil will be classified as the unsuitable material and for the purpose of construction; it will be removed and replaced by the other material with better strength and quality, which complies with the specified requirements. According to the correlation from the curve, it was not stated that when the moisture content of the clay soil reduced, the strength would increased. These correlations obtain by the range of moisture content used during the preparation of the compacted sample and it was impossible to compact the clay soil at the dry condition.

Table 4.11: Moisture content and CBR value for various types of soil

Soil description	Moisture content (%)	CBR (%)
Tg. Pelepas Marine Clay	12.67	28.39
	19.02	8.08
	26.89	3.47
	45.91	0.60
	56.05	0.23
UTM Yellowish Clay	45.21	8.46
	42.60	14.73
	36.27	16.63
	53.43	0.35
	59.11	0.23
Kulai Pinkish White Clay	22.20	9.77
	23.42	6.96
	29.95	0.62
	40.46	0.10
	42.09	0.07

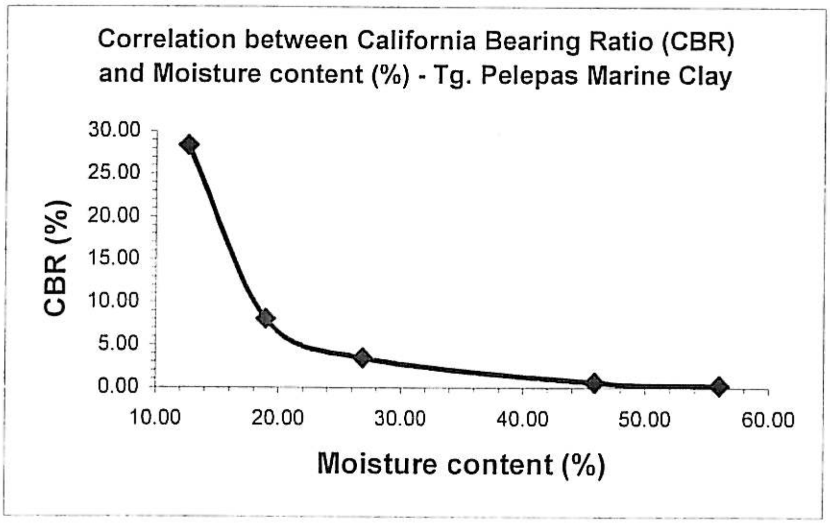


Figure 4.3: CBR (%) and moisture content (%) correlation for Tg. Pelepas Marine Clay

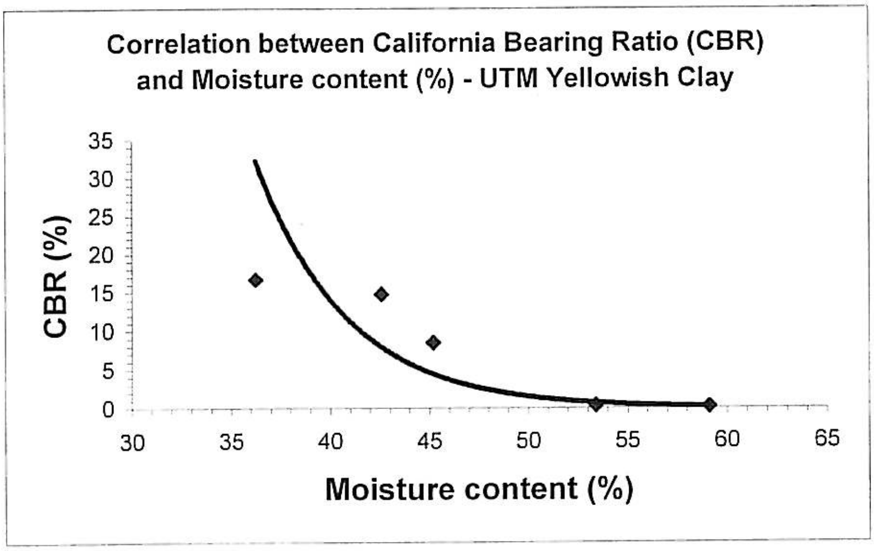


Figure 4.4: CBR (%) and moisture content (%) correlation for UTM Yellowish Clay

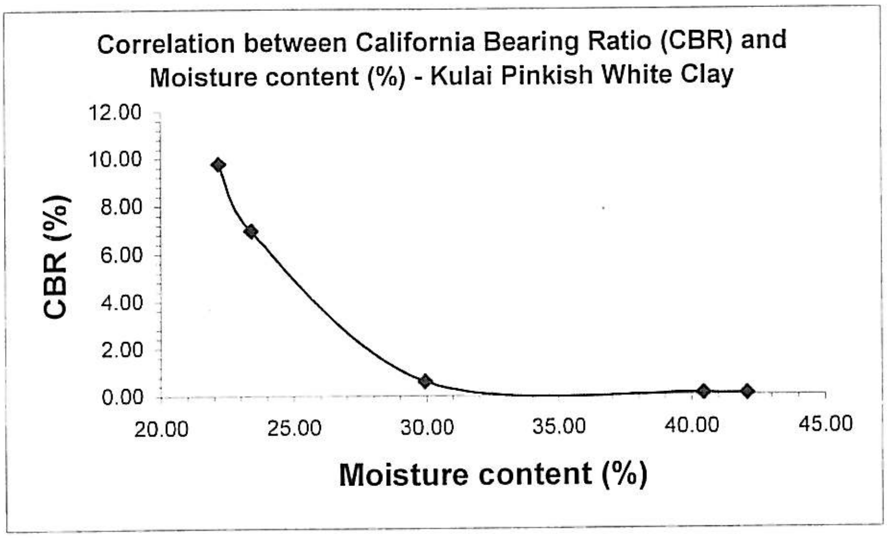


Figure 4.5: CBR (%) and moisture content (%) correlation for Kulai Pinkish White Clay

As discussed in Chapter III, the amount of water for soil-lime mixtures for this research was based on CBR value of the soil at 2 %. From the correlations, the moisture content of CBR value at 2 % for Tg. Pelepas is 30%, UTM is 48% and Kulai is 27 % respectively.

4.4.2 Unconfined Compressive Strength (UCS)

The result for the UCS tests for Tg. Pelepas Marine Clay, UTM Yellowish Clay and Kulai Pinkish White Clay were summarized and tabulated as shown in Table 4.12 respectively and **Appendix P** gives the details of the test. From the result, it was clearly showed that when the moisture content increased, the strength of the soil was reduced.

Table 4.12: Unconfined Compressive Strength (UCS) test for Tg. Pelepas Marine Clay, UTM Yellowish Clay and Kulai Pinkish White Clay

Soil description	UCS (kPa)	Moisture Content (%)
Tg. Pelepas Marine Clay	42.90	35.84
	28.20	37.79
	20.40	40.10
	14.01	44.67
	12.40	43.69
UTM Yellowish Clay	54.91	43.46
	35.00	46.73
	35.00	49.85
	22.10	49.38
	20.40	53.46
Kulai Pinkish White Clay	124.00	27.20
	76.00	29.60
	28.40	33.32
	44.90	32.72
	17.90	35.49

4.4.3 Vane Shear Strength (VSS)

Another type of strength test for the natural soil was laboratory vane shear strength test. From the test resulted as shown in Table 4.13, it was directly showed that the strength of the soil was reduced when the moisture content increased like the result of CBR and UCS test. All results from the CBR, UCS and VSS test were compiled, and then were manipulated to generate the correlation among the test.

Table 4.13: Vane shear strength for Tg. Pelepas Marine Clay, UTM Yellowish Clay and Kulai Pinkish White Clay

Soil description	Vane Shear Strength (kPa)	Moisture Content (%)
Tg. Pelepas Marine Clay	12.83	35.84
	21.88	37.79
	17.97	40.10
	12.06	44.67
	13.54	43.69
UTM Yellowish Clay	10.01	43.46
	27.85	46.73
	18.35	49.85
	15.59	49.38
	14.24	53.46
Kulai Pinkish White Clay	3.66	27.20
	16.24	29.60
	11.55	33.32
	12.64	32.72
	8.34	35.49

4.4.4 Correlation between California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) Test

According to the result of remolded soil strength test, several correlations between the tests involved can be achieved. This correlation might useful for the soil strength test knowledge. This stage is only the initial stage to develop the correlation among the test and it require more tests to get the best correlation and contribute to the soil strength testing knowledge.

The first correlation obtained is between the Vane Shear and Unconfined Compressive Strength (UCS) test. The curve of the remoulded soil strength is shown in Figure 4.6. From the curve, it was difficult to establish any correlation between both tests. The results then were normalized with the soil properties values to predict the correlation.

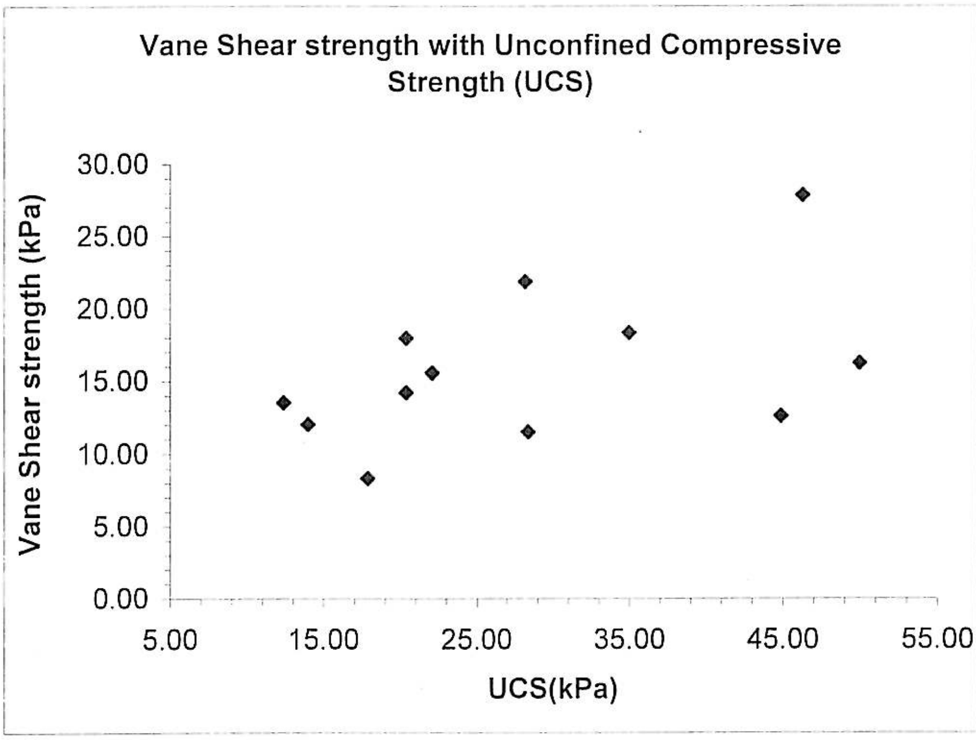


Figure 4.6: Vane Shear strength with Unconfined Compressive Strength (UCS)

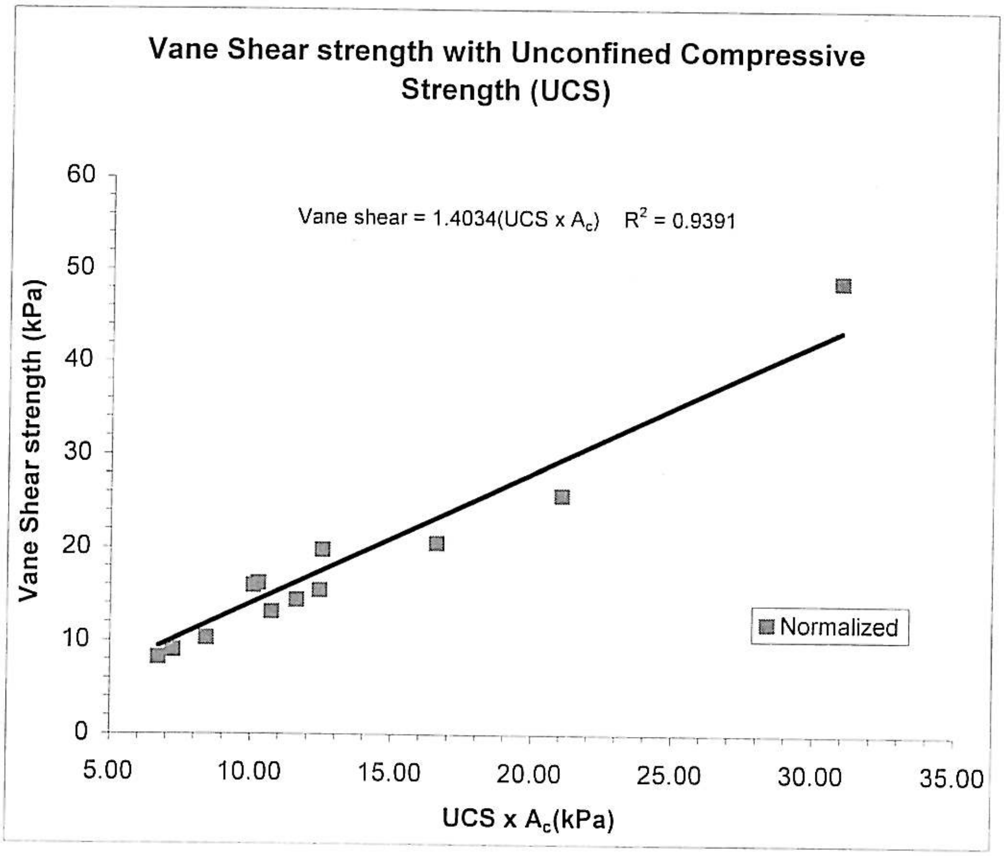


Figure 4.7: The curve of the correlation between Vane Shear strength and Unconfined compressive Strength (UCS) after normalization

The UCS value was normalized with the function of clay activity, A_c of the soil. As a result, after the normalization process, the data were merge linearly to give the best correlation for both tests and from the curves (Figur 4.7); the correlation of both tests is given as in Equation 4.4.

$$\text{Vane Shear Strength} = 1.4034\text{UCS x } A_C \quad (4.4)$$

Another correlation obtained during the remolded soil strength test is the correlation between CBR and UCS. According to the Equation 4.4, the value of the UCS can be determined. The curve of the remoulded strength is shown in Figure 4.8.

At the early stage, the data were tabulated inconsistency and as previous process, the CBR value was normalized with the clay activity of the soil, A_c. As a result, the curve as showed in Figure 4.9 give the best correlation between CBR and UCS. After the normalization, the data was merge linearly to give the best correlation and this correlation can be expressed as in Equation 4.5.

$$\frac{CBR}{A_C} = 0.0585 \times UCS \tag{4.5}$$

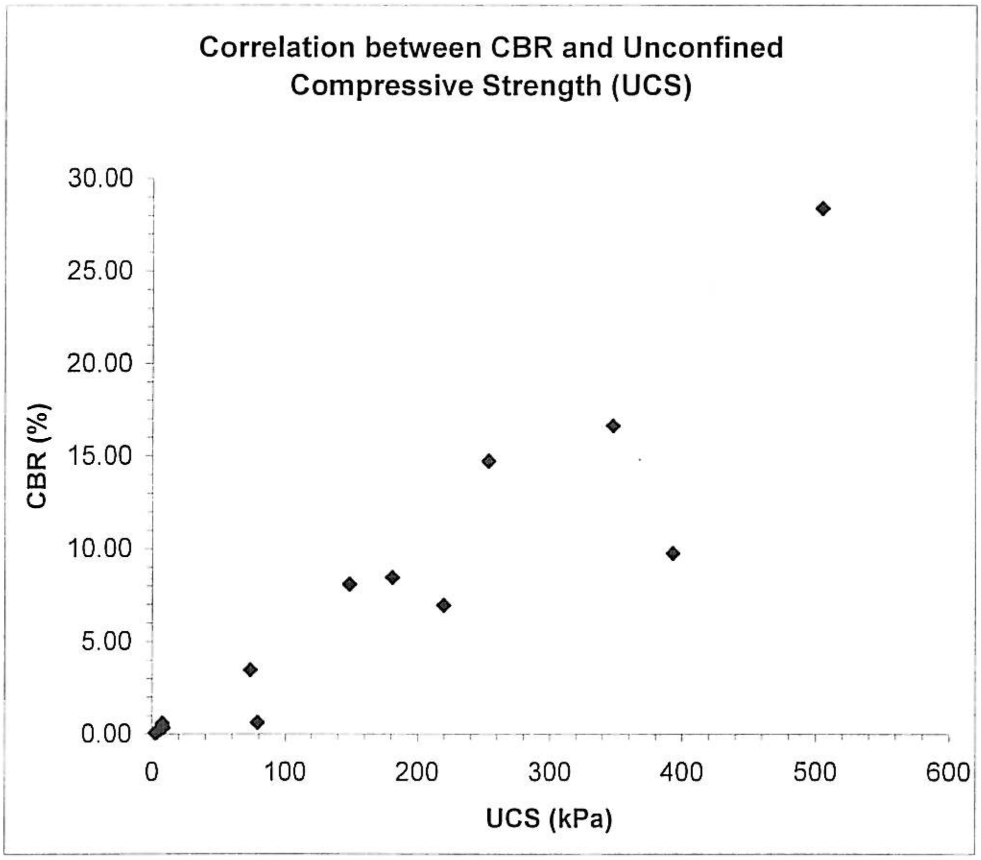


Figure 4.8: CBR with Unconfined Compressive Strength (UCS)

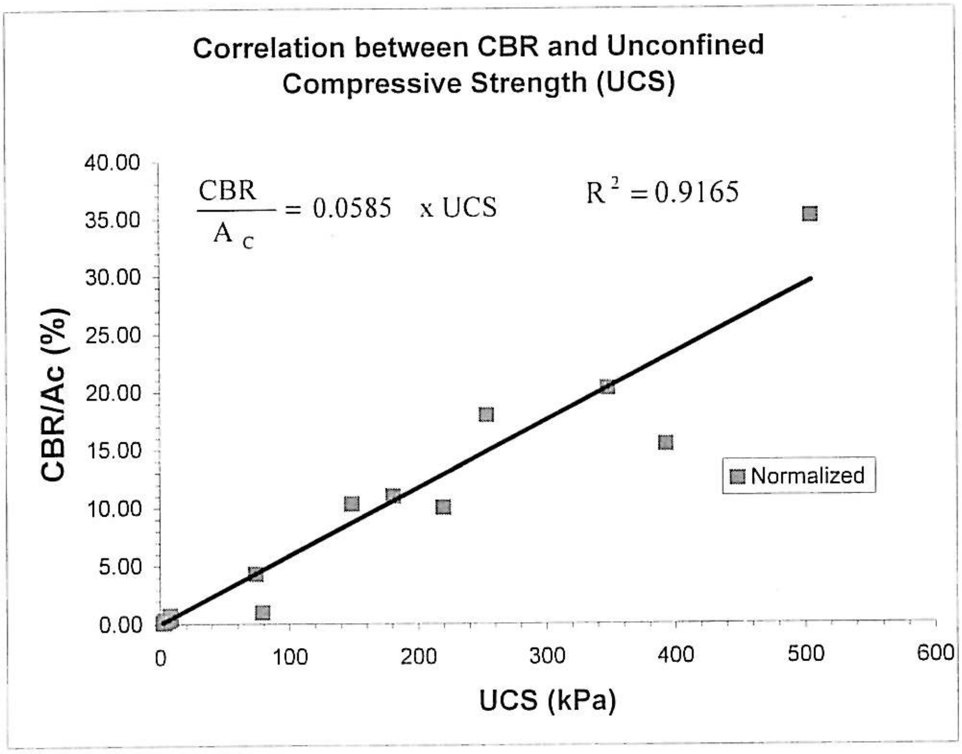


Figure 4.9: The curve of the correlation between California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) test

4.5 Strength Development Result

The quality of cement or lime stabilized materials is usually assessed on the basis of strength tests made on the material after the stabilizer has been allowed sufficient time to harden. The strength of the stabilized soils can be evaluated in many ways, of which the most popular are the Unconfined Compressive Strength (UCS) test for cement-stabilized soils and the California Bearing Ratio (CBR) test for lime-stabilized soils. Both have been critized on the ground that neither reflects the manner in which a stabilized layer is stressed in the road pavement. They are used mostly frequently because of the relative ease with which they can be performed. This makes them particularly suited to routine control where large number of test may be needed on a daily basis (Sherwood, 1993).

Table 4.14: The improvement of California Bearing Ratio (CBR) for various types of clays (after Bell, 1996)

Material	Optimum lime content	Optimum moisture content	Maximum dry density	CBR (%)
Kaolinite	0	29	1.40	1
	6	31	1.33	14
Montmorillonite	0	20	1.29	9
	4	25	1.15	18
Quartz	0	28	1.41	1
	6	32	1.40	22

The California Bearing Ratio (CBR) test is an empirical test developed by the California State Highway Department for the evaluation of sub-grade strength. For the clay soils, CBR value was improved by the addition of lime as shown in the Table 4.14. The strength of the clay soils increase more than 100 per cent after stabilized with the optimum lime content. The CBR value increase immediately after the addition of lime and it continues to increase with time if there is lime available in excess of the lime fixation point (Bell, 1996).

For the strength development in this research, Table 4.15, 4.16 and 4.17 gives the result of the CBR test for the stabilized soils with their curing period. From the result, Tg. Pelepas Marine Clay gives CBR value of 12.50 % at 7 days, which is increasing about 2 % from the value at 4 days. Then the strength increased to 14.77 % at 11 days before being reduced to 11.59 at 14 days. It was not much reduction of strength and it showed that the silicate gel from the chemical reaction was gradually starts to crystallize. Then, the strength at 31 days was increased to 14.77 % and 26.74 % for 56 days. For UTM Yellowish Clay, there was increasing of strength at the initial stage of stabilization as shown in Table 4.16. CBR value at 4 days was 2.80 % and the strength increased rapidly about 4 times at 9 days, which give the CBR value of 9.10 %. With allowing more time of curing, the strength increased at 14, 28 and 56 days with the CBR value of 10.77 %, 11.75 % and 14.24 %.

Table 4.15: California Bearing Ratio (CBR) results of lime-stabilized soil with 4 days soaking period – Tg. Pelepas Marine Clay

Sample	Age	Soaked CBR (%)	ΔCBR
Without lime	4	2.00	0
Tg. P A	4	6.29	4.29
Tg. P 0	7	12.50	6.21
Tg. P 5	11	14.77	2.27
Tg. P 4	14	11.59	-3.18
Tg. P 3	31	14.77	3.18
Tg. P 2	56	26.74	11.97

Table 4.16: California Bearing Ratio (CBR) results of lime-stabilized soil with 4 days soaking period – UTM Yellowish Clay

Sample	Age	Soaked CBR (%)	ΔCBR
Without lime	4	2.00	0
UTM A	4	2.80	0.80
UTM 5	9	9.10	6.30
UTM 4	14	10.77	1.67
UTM 3	28	11.75	0.98
UTM 2	56	14.24	2.49

Table 4.17: California Bearing Ratio (CBR) results of lime-stabilized soil with 4 days soaking period – Kulai Pinkish White Clay

Sample	Age	Soaked CBR (%)	△CBR
Without lime	4	2.00	0
Kulai A	4	10.15	8.12
Kulai 0	7	20.63	10.48
Kulai 5	12	17.50	-3.13
Kulai 4	15	15.00	-2.50
Kulai 1	33	27.65	12.65
Kulai 2	56	40.15	12.50

For Kulai Pinkish White Clay, there was rapidly gained in strength when CBR value increased to 10.15 % from 2.0 % within 4 days of curing period. The strength continued to increase and at 7 days, CBR value achieved 20.63 %. However, the strength of the stabilized soil was reduced after 7 days. At the age of 12 days, the CBR value was reduced to 17.50 % and 15.00 % at 15 days. This trend is similar to the Tg. Pelepas Marine Clay. However, two weeks after the stabilization process, the soil strength gradually to increase with time. At 33 days, the CBR value is 27.65% and at the 56 days, the CBR value is 40.15%.

The overview of the whole results is shown in Figure 4.10. For all samples, the trend of the strength development was increased with the curing period. From the curve, the CBR value of lime-treated soil was developed rapidly especially in the initial stage. Lime has an almost instantaneous effect on the plasticity of a clay and therefore upon the strength. As shown in Figure 4.11, the strength of the soil-lime mixture increases rapidly after 6 minutes of the mixing process. It was due to the reduction of the moisture content in the soil. A pozzolana material from the clay minerals is capable to react with lime in the presence of water, at the ordinary temperatures, to produce cementitious compounds. The highly alkaline environment (pH 12.4) produced by the addition of lime promotes the dissolution of the clay, particularly at the edges of the clay plates, permitting the formation of calcium silicates and aluminates. Theses comentitious products are broadly similar in composition of those of cement paste.

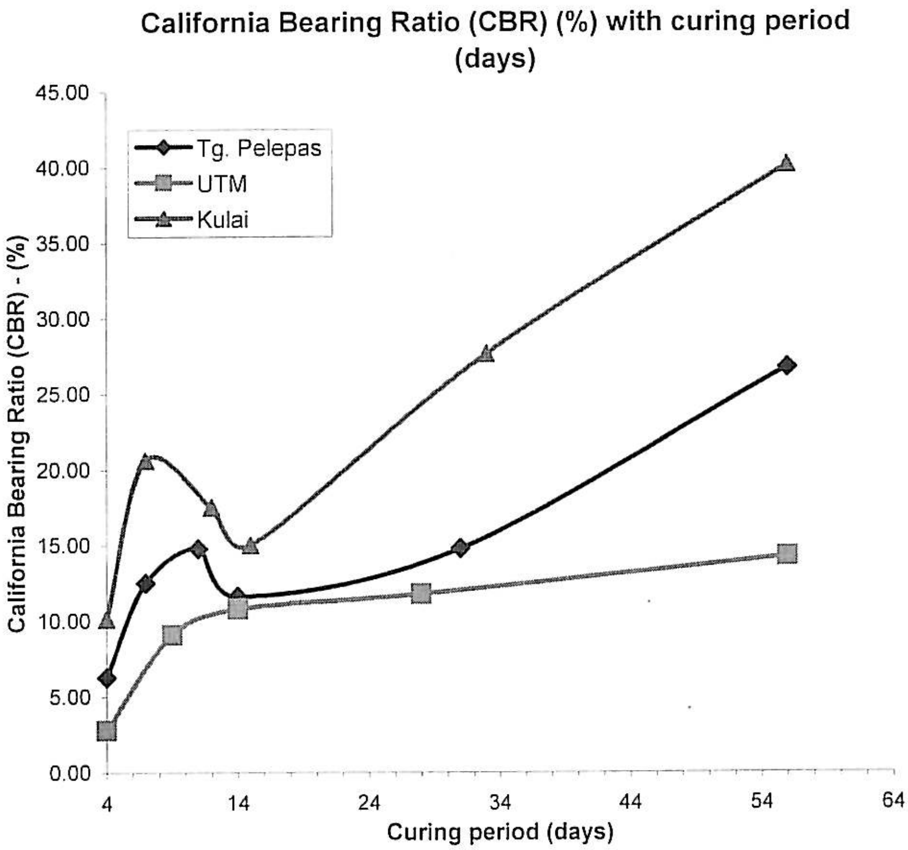


Figure 4.10 CBR (soaked) value with the curing period for various types of stabilized soils

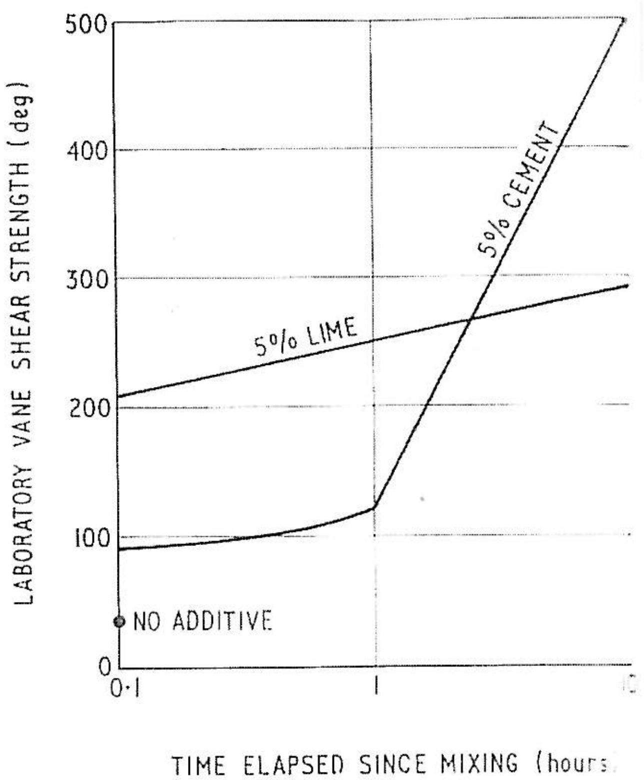


Figure 4.11: The rapid initial increase of strength of lime-stabilized clay compared with cement-stabilized clay (after Ingles and Metcalf, 1972)

However, the first two weeks, the increased pattern of strength was not too clear. It is due to the pozzolanic reactions, which may not completely react. The process is relatively slow because the available lime has to diffuse through both the soil structure and the initial cementitious products to the reaction sites (Sherwood, 1993). Therefore, the silicates and aluminates gel were not complete crystallized to form an interlocking structure. After two weeks, the increased pattern of strength was clear and it shows that the full stabilization process was starts to develop. The effects of lime on the CBR value increases with time as the pozzolanic reaction take effect as shown in Figure 4.12. It is generally considered that the slow cementation process in lime stabilization permits a relatively long time for stabilized soil developed the strength. The strength will increase to a certain period when there is no more development of strength occurs.

Little (1996) mentioned that extended curing either in the laboratory or under field conditions may produce strength increases of several hundred psi. Field data indicate that some soil-lime mixture strength continues to increase with time up to in excess of ten years. Working example for the CBR test is given in **Appendix Q**.

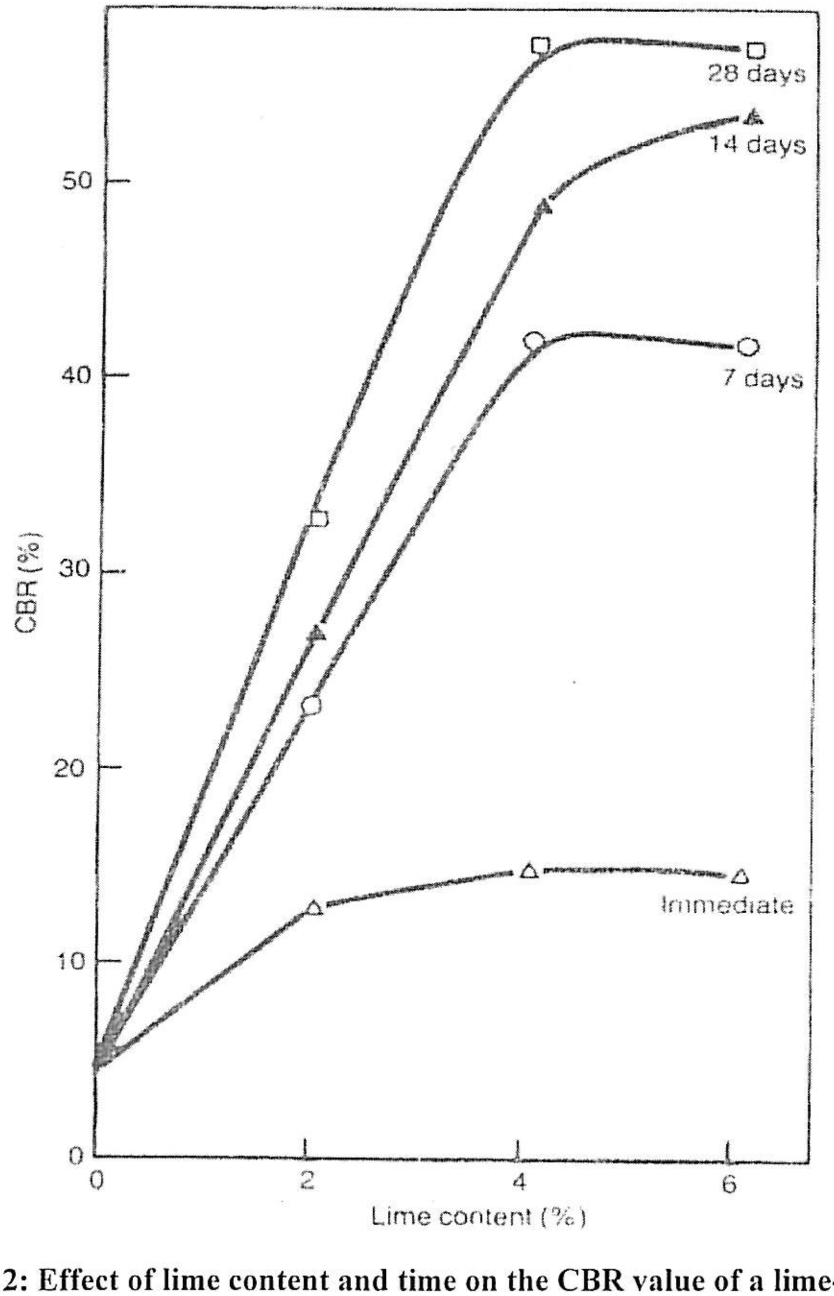


Figure 4.12: Effect of lime content and time on the CBR value of a lime-stabilized soil (after Sherwood, 1993)

4.6 Result of the Model Testing for Materials Tested

4.6.1 Deformation Model

The performance of stabilized layer in reducing the settlement and reducing andulation on road surface was observed from the deformation model testing. As discussed in Chapter II, the magnitude of the settlement for the beneath layer of pavement was influenced by the deflection of the road pavement. Therefore, the strong and stiff sub-grade will reduce the amount of deflection of the road pavement beside the quality of the materials of the pavement itself. If the sub-grade is weak, the surface of the pavement may not smooth and not convenient to ride on it. By lime stabilization method, it provided a stiff layer for the placement and compaction of the sub-base or road base for the road construction.

From the result, it was clearly observed that load which can be sustained by the sub-grade increased after the lime stabilization method is applied as a capping layer. From the Table 4.18, the unstabilized sample gives a value of 0.10 kN of load. For stabilized samples, the load of 1.00 kN for an immediate sample and 1.00, 1.60 and 4.00 kN for sample with curing period of 7, 14 and 28 days respectively.

Table 4.18: The value of failure load (kN) for unstabilized and stabilized sample

Failure Load (kN)				
Without lime (Control)	**With lime**			
	Curing Period (Day)			
	0	7	14	28
0.10	1.00	1.00	1.60	4.00

Table 4.19: The settlement (mm) for unstabilized sample

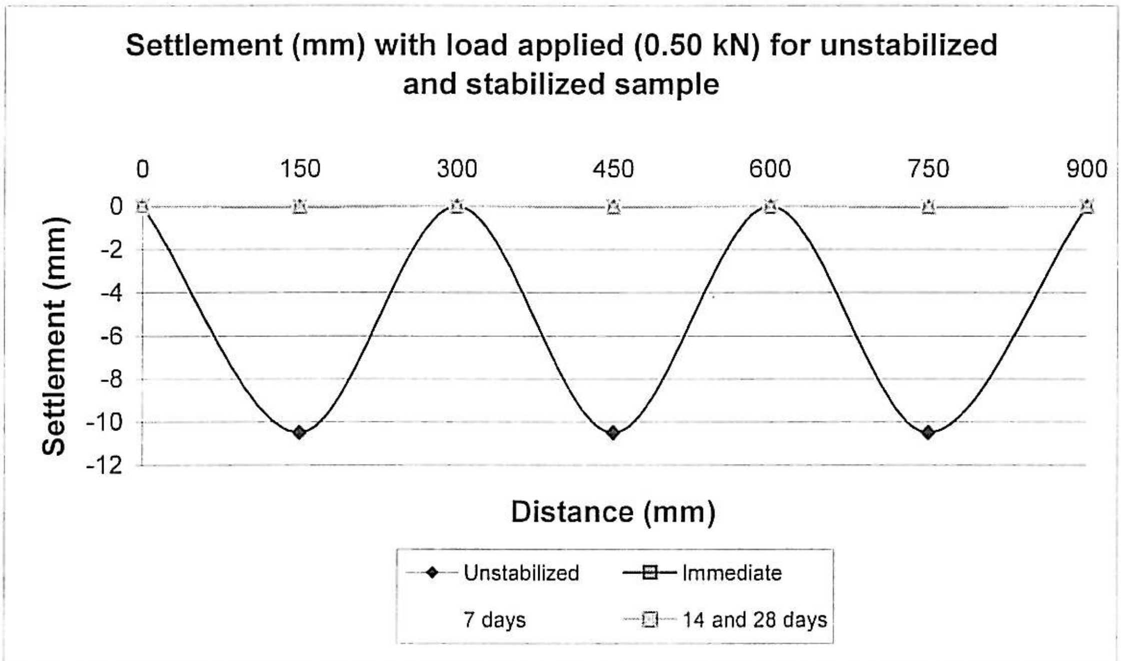
Sample	Applied load (kN)		
	0.50	**1.00**	**1.50**
Without lime (control)	10.50	16.00	20.00

Table 4.20: The settlement (mm) for stabilized samples

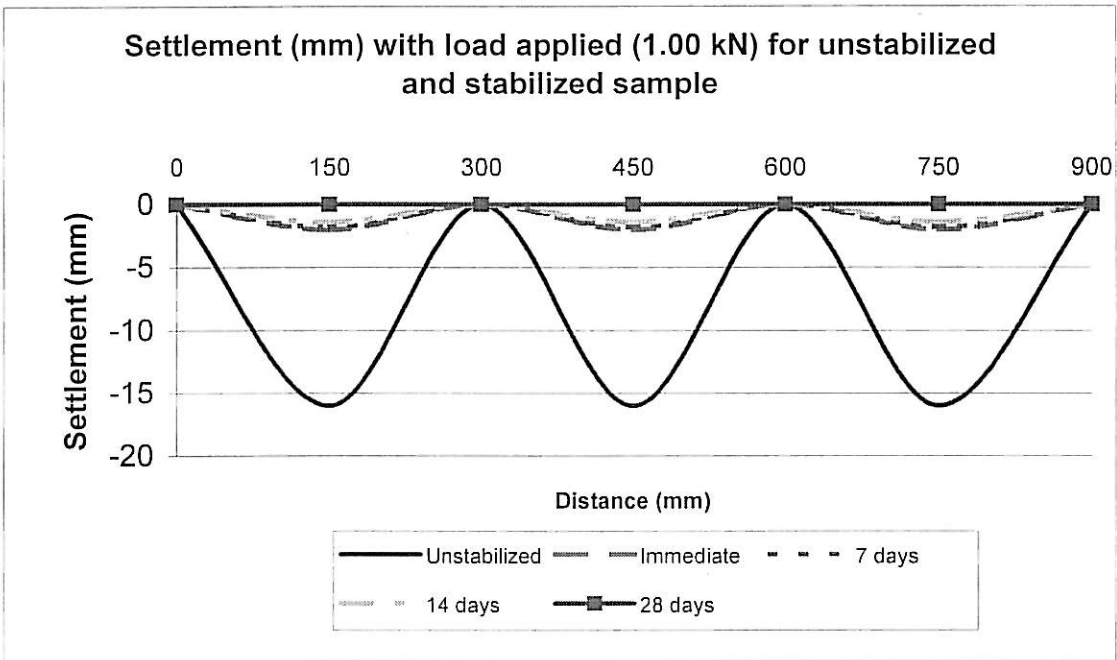
Curing period (days)	Applied load (kN)		
	0.50	**1.00**	**1.50**
0	0.04	2.00	4.80
7	0.02	1.80	4.60
14	0.00	1.40	2.50
28	0.00	0.01	0.02

Besides the increasing of the load that can be sustained by the stabilized capping layer, the results also indicated that the settlement occur at the sub-grade surface was reduced after providing lime stabilized soil as a capping layer. In order to express the finding, the settlement of the sample was measured for both unstabilized and stabilized samples at the three values of loads, which have been applied. Firstly, the settlement was measured at 0.5 kN of load and followed at 1.0 kN and 1.5 kN respectively. For the unstabilized sample, the settlement values of 10.50 mm, 16.00 mm and 20 mm for the applied load of 0.5 kN, 1.0 kN and 1.5 kN respectively. However, after stabilization, the settlement was reduced about 100 per cent for 0.50 kN, which only 0.04 mm settlement occur. About 88 percent reduction of settlement occur for 1.0 kN of load when only 2.00 mm of settlement was measured. For 1.5 kN of loading, the settlement reduced to about 76 per cent with 4.80 mm of settlement was measured.

However, with allowing extended time of curing period, the percentage of settlements reduction was increased. For 0.50 kN of load, the settlement reduced for 99.60 % at immediate sample, 99.8% for 7 days. For 14 and 28 days, there was no settlement occurred with 0.5 kN of applied load. For 1.0 kN of load, the settlement reduced for 88 % for immediate sample, 89% for 7 days, 91.25 % for 14 days and 99.90 % for 28 days. For 1.5 kN of load, the settlement was reduced for 76 % for immediate sample, 77 % for 7 days, 88 % for 14 days and 99.98 % for 28 days. It showed that just a little settlement value for the stabilized sub-grade when allowed more time of curing period as shown in Figure 4.13. All the settlements value obtained from the test were shown in Table 4.19 and 4.20.



(a)



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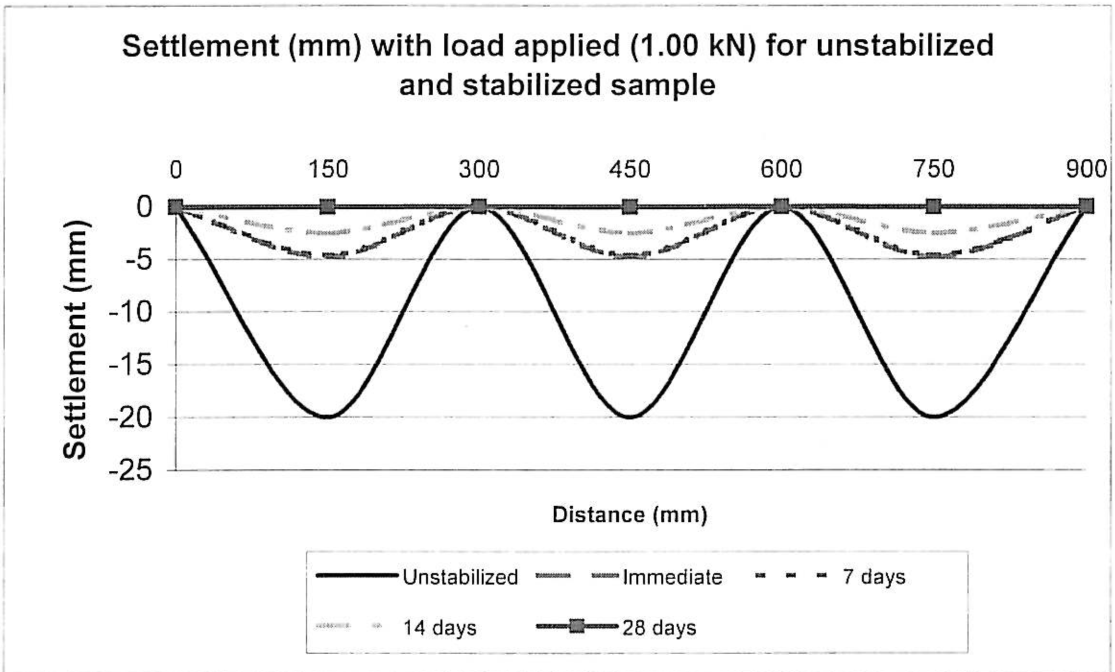


(c)

Figure 4.13: The pattern of wavy condition with unstabilized and lime-stabilized capping layer for road construction (Load applied at (a) 0.5 kN (b) 1.0 kN (c) 1.5 kN)

4.6.2 Bearing Capacity Model

A variety of soils are frequently present along any highway corridor before construction and different bearing strength exist when different types of soils are used to construct pavement sub-grade. To avoid bearing capacity failures during construction, certain minimum sub-grade strength must exist to sustain constructuion traffic. When the actual sub-grade strength is smaller than the minimum strength required sustaining construction traffic, it should be stabilized the sub-grade to increase the strength. After the stabilization, strength of the treated and untreated layers must be selected for the design analysis. If the improved strength created by chemical stabilization is ignored, the pavement thickness obtained from the design analysis may be too conservative (Hopkins *et.al*, 1994).

4.6.2.1 Model Bearing Capacity ($q_{u\ (model)}$)

Based on the model testing, the bearing capacity of the stabilized and unstabilized soil was obtained. As defined from the previous section, the total allowable settlement for road was 20 mm and for the model justification, the value of settlement was 2 mm after considered the scaling factor of 10 during the model construction. Then the failure load was determined due to the settlement of 2 mm for each sample. Table 4.21 shows the value of failure load for the bearing capacity model testing.

From the result, the failure load increased in term of the curing period and the thickness of the stabilized soil factor. With allowing extended time of curing, the stabilized soil showed the gradual development of strength in compacted soil-lime mixtures. The slower, long-term reaction resulting in formation of the final cementitious products contributes to this development (Wood, 1988).

Table 4.21: Failure load (kN) for the bearing capacity model testing

Samples of bearing capacity models	B Min	B Max
H0	2.40	1.60
H4D0	3.60	4.00
H6D0	5.00	4.00
H8D0	5.40	7.00
H4D7	3.00	4.00
H6D7	7.00	6.00
H8D7	9.50	9.00
H4D14	3.00	5.00
H6D14	9.00	9.00
H8D14	14.00	12.00
H4D28	5.00	7.00
H6D28	10.00	10.00
H8D28	16.00	16.00

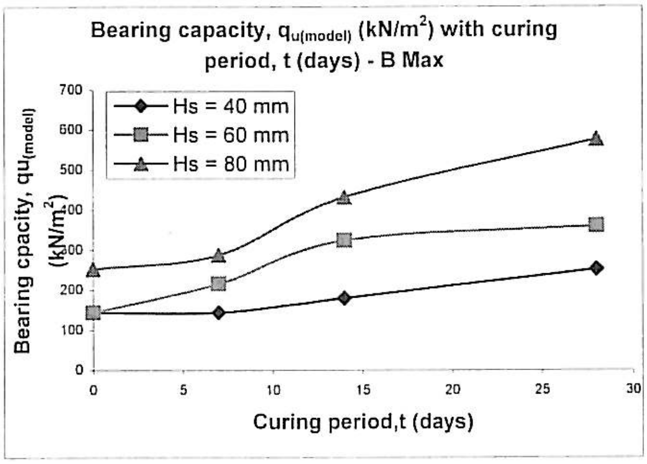
The bearing capacity of the model was calculated based on Equation 4.6. There were two values of bearing capacity obtained. Firstly, the bearing capacity value based on the minimum contact area (B Min), which contact to the surface of the stabilized layer and secondly from the maximum contact area (B Max). These two values of contact area were described in Chapter III (section 3.8).

$$q_{u(model)} = \frac{\text{Load (kN)}}{\text{Area (m)}^2} \tag{4.6}$$

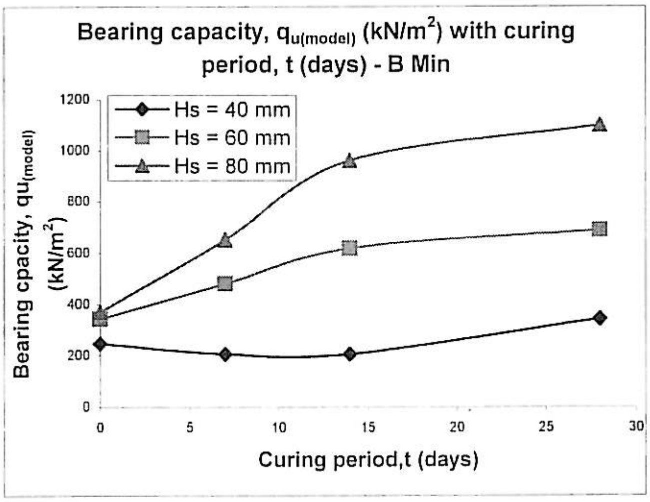
According to Bell (1988), the strength of the lime-stabilized soil depends on the curing period and the percentage of lime used during the mixing process (Figure 2.6). From Equation 4.6, the bearing capacity of the model was calculated and the result showed that the strength of the stabilized soil, which is dependent on curing period and the thickness of the stabilized layer. When the thickness of the stabilized layer increased, the bearing capacity of the structure also increased. Table 4.22 gives the value of bearing capacity from the model testing and Figure 4.14 shows the pattern of increasing strength with the increased thickness of the stabilized layer.

Table 4.22: Bearing capacity, $q_{u(model)}$ (kN/m^2) for the model testing

Samples of bearing capacity models	B Min	B Max
H0	164.95	57.66
H4D0	247.42	144.14
H6D0	343.64	144.14
H8D0	371.13	252.25
H4D7	206.19	144.14
H6D7	481.10	216.22
H8D7	652.92	288.29
H4D14	206.19	180.18
H6D14	618.56	324.32
H8D14	962.20	432.43
H4D28	343.64	252.25
H6D28	687.29	360.36
H8D28	1099.66	576.58



(a)



(b)

Figure 4.14: The bearing capacity, $q_{u(model)}$ increased with the increased thickness of stabilized layer (a) B Max (b) B Min

4.6.2.2 Bearing Capacity (Theory) ($q_{u(theory)}$)

Beside the actual bearing capacity of the model, which is directly obtained from the estimated load and contact area of the stabilized layer, the bearing capacity, q_u based on the theory were also calculated. As mentioned by Hopkins *et.al*, (1994), the treated and untreated layer should be considered during the design analysis for the stabilized sub-grade. Equation 2.4 and 2.5 proposed by Braja (1999), suggested the theory for bearing capacity of the strong clay soil underlain by the weaker clay soil. The undrained shear strength of the lime-stabilized soil depends to the curing period. Holt and Freer-Hewish (1996) mentioned that by increased the curing for the lime stabilized soil is likely to produce strength gains. Lime is an effective stabilizer of clays and it will alters the ability of the clay to hold water at its surface and can react with the clay to produce a cementing agent to gain strength (Little, 1996). The strength of the soil-lime mixture also increases due to the development of a cemented matrix among the soil particles. Based on work done by Tan Sze Nee, 2000, the undrained shear strength of the lime-stabilized of UTM Yellowish clay is shown in Table 4.23. From the model testing, the undrained shear strength of the stabilized layer was also measured and the result obtained is shown in Table 4.24. From both results, the correlation between the undrained shear strength of the stabilized soil and the curing period was established and can be expressed using Equation 4.7. The value of the strong clay was calculated using this equation to determine the bearing capacity of the structures. The undrained shear strength of the stabilized soil also depends with the thickness of the stabilized layer. When the thickness of the soil-lime mixture increase, the stiffness and load spreading capability of the stabilized soil also increase and it is contributed to the increasing of the bearing capacity value. As a result, the bearing capacity of combination structure, lime-stabilized capping layer underlain soft clay increased with time and the thickness of the stabilized layer.

$$C = 5.7622t + 51.303 \tag{4.7}$$

The increasing value bearing capacity obtained is shown in Table 4.25. Figure 4.15 shows the pattern of bearing capacity value based on the theory. As a result for the model testing, the bearing capacity value from the theory of the lime-stabilized layer underlain the soft clay will increase with the curing period of the soil-lime mixture and the thickness of the stabilized layer factor.

Table 4.23: The undrained shear strength of the lime-stabilized UTM Yellowish Clay (after Tan, Sze Nee, 2000)

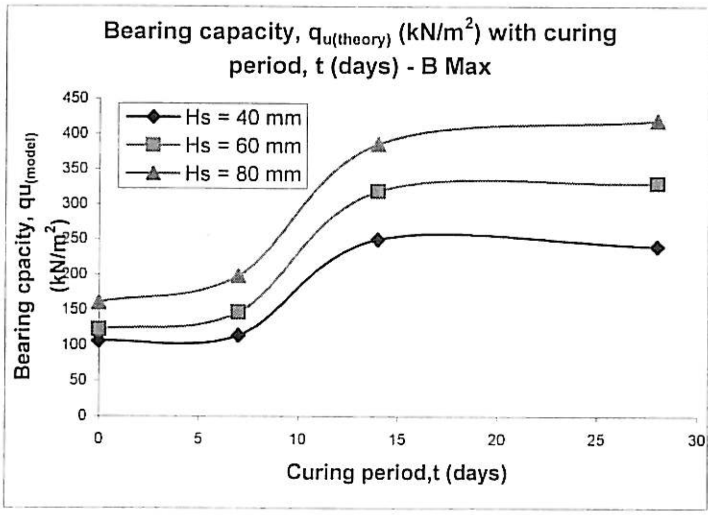
Curing period, t (days)	Undrained shear strength, C (kN/m^2)
0	9.79
7	54.00
14	110.87
28	197.91

Table 4.24: The undrained shear strength of the lime-stabilized UTM Yellowish Clay for the bearing capacity model.

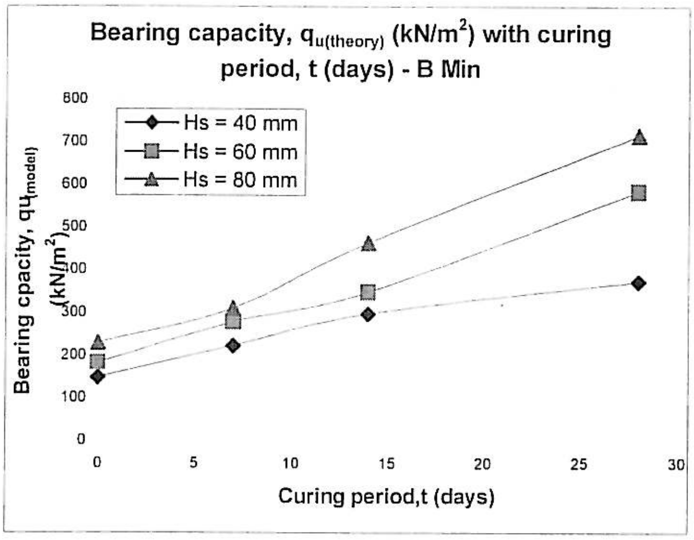
Sample	Curing period, t (days)	Undrained shear strength, C (kN/m^2)
BMXH8	0	76.00
BMNH8	0	90.00
BMXH8	7	80.00
BMNH8	7	95.00
BMXH8	14	90.00
BMNH8	14	200.00
BMXH8	28	234.00
BMNH8	28	215.00

Table 4.25: Bearing capacity, $q_{u(theory)}$ (kN/m^2) for the model testing

Samples of bearing capacity models	B Min	B Max
H0	59.85	43.49
H4D0	149.59	106.47
H6D0	183.11	122.49
H8D0	230.20	160.69
H4D7	223.66	114.22
H6D7	279.77	147.07
H8D7	311.14	198.43
H4D14	298.56	249.92
H6D14	349.68	318.84
H8D14	464.22	386.27
H4D28	372.97	240.02
H6D28	584.94	330.89
H8D28	716.94	419.36



(a)



(b)

Figure 4.15: The bearing capacity, $q_{u(theory)}$ increased with the increased thickness of stabilized layer (a) B Max (b) B Min

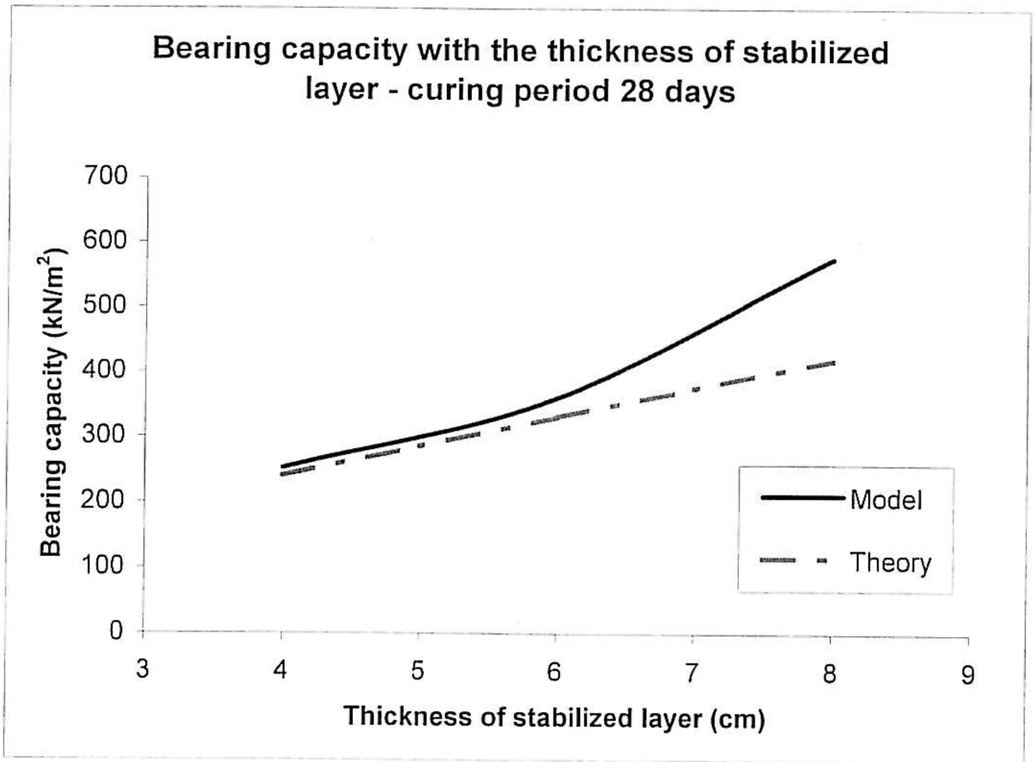


Figure 4.16: Bearing capacity of the lime-stabilized soil underlain soft clay layer at curing period 28 days

For the design purposes, the strength of the lime-stabilized soil was obtained at the age where it showed the stable condition in strength development. From the strength development result, it was proposed that the strength at curing period of 28 days was used for the design. Figure 4.16, it shows the curve for the bearing capacity of the experiment and theory value. From the figure, the bearing capacity curve for the experiment value showed similar pattern to the theory value. Therefore, it is possible to use the bearing capacity from theory to calculate the actual bearing capacity of the lime-stabilized soil underlain soft clay layer.

4.7 Proposed Weight Limit of the Road

4.7.1 Introduction

Stabilization, as applied to a pavement construction, can be defined as a means of permanently increasing their strengths and bearing capacity. In order to achieve stability, an additive must be incorporated into the soil or base material. With clayey soil, the most effective and commonly used stabilizer is lime. Lime will react both physically and chemically to yield a stable paving material. There are many methods for designing pavement thickness. Usually, most methods are formulated around a 'layer cake' system where it is intended that each layer decrease stress for a deeper placement to prevent overstressing (McDowell, 1972).

Based on this statement, the maximum loading from vehicles before the sub-grade is overstressed can be determined. The bearing capacity of lime-stabilized layer underlain a soft clay layer was investigated and the physical model was developed to illustrate such a condition. The bearing capacity of the structure was obtained by the model testing and the improvements have been discussed in section 4.6. From the results, the curve of the bearing capacity of the model showed a similar pattern to the theory at curing period of 28 days. Therefore, the bearing capacity of the structure was calculated using the equation proposed from the theory. With the consideration of the factor of safety, the weight limit of the road constructed on stabilizing capping layer was proposed. Due to stabilization, the thickness of the road is reduced due to the increase in the CBR of the stabilized sub-grade.

4.7.2 Weight Limit Design Criteria

According to the Road Note 31 (1993) there are three main steps to be followed in designing a new road pavement. The steps are:

i. Estimating the amount of traffic and cumulative number of equivalent standard axles that will use the road over the selected design life.
ii. Assessing the strength of the sub-grade soil over which the road is to be built.
iii. Selecting the most economical combination of pavement materials and layer thickness that will provide satisfactorily service of the design life of the pavement.

However, all the steps mentioned above can be achieved with different strategies. Several different methods may be used to design the flexible pavement layer. Flexible pavement design method usually falls into one of several categories: (1) empirical methods, (2) limiting shear failure methods, (3) limiting deflection methods, (4) regression methods based on pavement performance or road test or (5) mechanistic – empirical methods (Little, 1995). However, this research proposes to estimate weight limit of the road in order to protect the sub-grade from being overstressed by the load from vehicles. The actual stress on the sub-grade surface was estimated based on the load from the vehicle and the contact area, B_{eff} (Figure 4.17). This contact area was dependent on the value of vehicle's tyre width, road thickness and load-spreading angle of the road materials. For design purposes, the width of the commercial vehicle (275 mm) was considered in the calculation. The bearing capacity of the lime-stabilized soil and soft clay was measured using the Equations 2.4 and 2.5. The lime-stabilized capping layer underlain soft clay was considered as a foundation before the road was constructed. As a foundation, the bearing capacity of the structure (q_u) was compared to the actual stress (q_n) developed from the vehicle at the road surface to determine its factor of safety (F.S). Based on a safety factor of 3, the maximum load for the specified road thickness and road materials was determined.

4.7.3 Assumptions of Weight Limit Design

The concept of the weight limit design was based on the theory of load distribution under soil. During the calculation of the maximum loading allowed for the road, several assumptions have been made. The material of the road was assumed as an isotropic and homogenous layer, therefore, the soil properties of the material were considered to be the same. Based on the Figure 4.17, the load distribution angle was equal to a gradient of 2:1. However, this gradient is the normal value for the load distribution angle which occurs in the soil. Therefore, the other assumption made during the weight limit design process is that the load distribution angle is dependent on the stiffness of the road materials. If the stiffness of the material is high, then the angle of load distribution increases. For comparison purposes, stiffer materials having load dispersion angles higher than normal are also analyzed. The other assumption is the load from the vehicle was totally transferred to the sub-grade surface during the calculation of the actual stress.

4.7.4 Design Concept

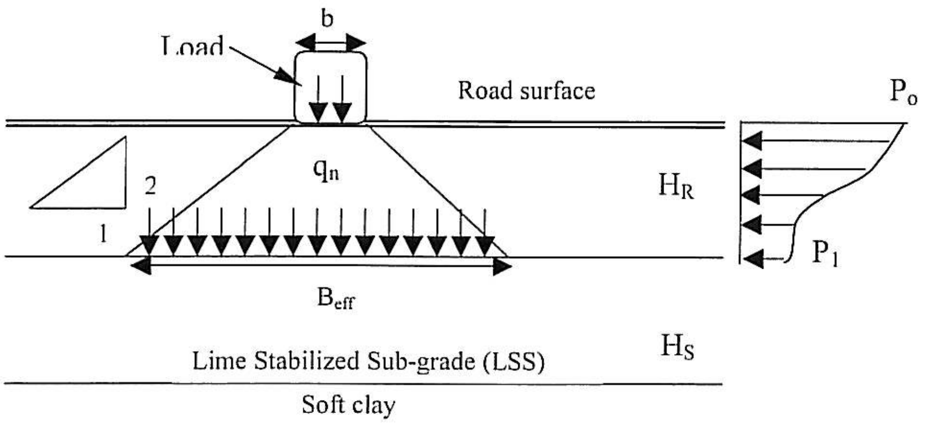


Figure 4.17: The concept for the road thickness design based on lime-stabilized capping layer underlain by soft clay layer

The design concept is based on the structural function of the pavement being to transfer the traffic load to the sub-grade without exceeding the strength of the sub-grade itself. From Figure 4.17, the wheel load is transmitted to the pavement through the tyre at an approximate pressure, P_o. The pavement distributes the wheel load over a larger area so that the maximum pressure on the sub-grade is only P_1. The design process involves the selection of pavement materials and thicknesses such that P_1 is within the sub-grade capacity.

As a foundation structure, the sub-grade must have enough strength to sustain this stress to avoid settlement. Settlement of the sub-grade causes structural deformation of the road which includes wheel tracking; rutting, crazing and potholing (Wignall *et.al*, 1991). According to Figure 3.17, the magnitude of settlement is considered to be the same for each layer of the road structure. Based on this situation, the sub-grade has limiting value of the settlement before failure occurs.

From Figure 4.17, the actual stress from the vehicle load on the lime-stabilized sub-grade surface is shown as q_n. As describe in section 4.7.2, the maximum load for the road was determined based on the bearing capacity of lime-stabilized and soft clay structure between the actual stresses, q_n. The factor of safety (F.S) could be defined in term of the ultimate bearing capacity, q_u as shown in Equation 4.8.

$$F = \frac{q_u}{q_n} \tag{4.8}$$

where, in the purpose for this research, q_n is the actual stresses from the vehicle load distributed to the lime-stabilized sub-grade surface and q_u is the bearing capacity of the lime-stabilized soil underlain the clay soil, which can be calculated using Equations 2.4 and 2.5. According to Figure 4.17, the actual stress from the load of the vehicle tyre can be calculated using the Equation 4.9.

$$q_n = \frac{\text{Load(kN)}}{\text{Area}} \tag{4.9}$$

$$q_n = \frac{\text{Load(kN)}}{B_{eff} \text{ x L}}$$

$$q_n = \frac{\text{Load(kN)}}{(b + H_R) \text{ x L}} \quad \text{for load spreading angle 2:1}$$

where, b is the maximum width of the vehicle tyre

H_R is the road thickness

L is the length of tyre imprint

From Equations 4.8 and 4.9, the maximum load capacity of the road can be calculated with the design factor of safety. The road thickness, H_R, varies from 0 to 1 m, and the load-spreading angles used are 2:1, 1:4 and 1:6 to represent the different stiffnesses of the road pavement materials. The factor of safety of 3 was considered for the foundation layer which sustain the dynamic loading. Finally, the design charts, which incorporate the maximum load capacity for the road and road thickness was produced. The example of the calculation is described in section 4.7.6.

4.7.5 Weight Limit Design

The weight limit of the road for the specified thickness of the stabilized soil can be obtained from the design charts. In order to use these charts, some parameters need to be assumed. The parameters of natural soil properties and strength, C_2 and for the stabilized soil, C_1 are determined. Then, the type of road materials used in design is determined and it will give the load-dispersion angle of the road structure. With the estimated thickness of capping layer, H_s and road thickness, H_R the weight limit of the road is obtained.

4.7.5.1 Natural Soil Condition

The initial stage of design is to determine the strength and properties of the natural (field) soil. This is to ensure the suitability of the soil for lime stabilization. If the PI value of the soil is above 10 % and the clay fraction is more than 10 %, the soil is suitable for lime stabilization (McDowell, 1972). Besides that, the strength of the natural soil, C_2 is also measured and should have a CBR value of at least less than 2 %. This is the minimum value of the sub-grade for the road thickness design. After the suitability of the soil has been determined, the stabilization process is carried out according to the specification, and the strength of the stabilized layer was measured.

4.7.5.2 Stabilized Soil Strength

For soils, which have been stabilized with lime, the strength increases and depends on the curing period. The strength of the stabilized soil, C_1 is measured and in this research, the correlation of the compressive strength of the stabilized soil with the curing period is shown in Equation 4.7. Then, the ratio between the soft layer, C_2 and stiff layer, C_1 is calculated.

4.7.5.3 Road Materials

The road materials contribute to the degree of load-spreading. If the materials become stiffer, the load-spreading angle increases. For the purpose of comparison, the load-spreading is taken to be 2:1 to represent natural soil condition. For stiffer road material on the other hand, the load-spreading angles considered are 1:4 and 1:6.

4.7.5.4 Thickness of Stabilized Layer, H_s (Capping)

Sherwood (1993) proposed that the stabilized layer which acts as the capping layer to have a thickness of 600 mm if situated above sub-grade having CBR values ≤ 2%. From the test results obtained from bearing capacity model, it can be shown that the thickness of the stabilized layer contributes to an enhanced bearing capacity of the sub-grade. Therefore, for design purposes, the thicknesses of the stabilized soil layer, H_s is to be in the neighbored of 600 mm. Hence, the values of H_s used in this research are 400 mm, 600 mm and 800 mm.

4.7.5.5 Estimated Road Thickness (H_R)

The estimation of road thickness, H_R, in this is based on practical values obtained in the field. From Road Note 31, the maximum thickness of the road which can be obtained from the catalogue is 1 m. Therefore, the maximum thickness of the road considered in order to produce the design charts is also 1 m. The design charts are produced for road thickness, H_R values of between 0 to 1 m, load-spreading angles of 2:1, 1:4 and 1:6, lime-stabilized capping layer thicknesses of 400, 600 and 800 mm and a safety factor of 3. The summarization of the design steps is shown in Figure 4.18.

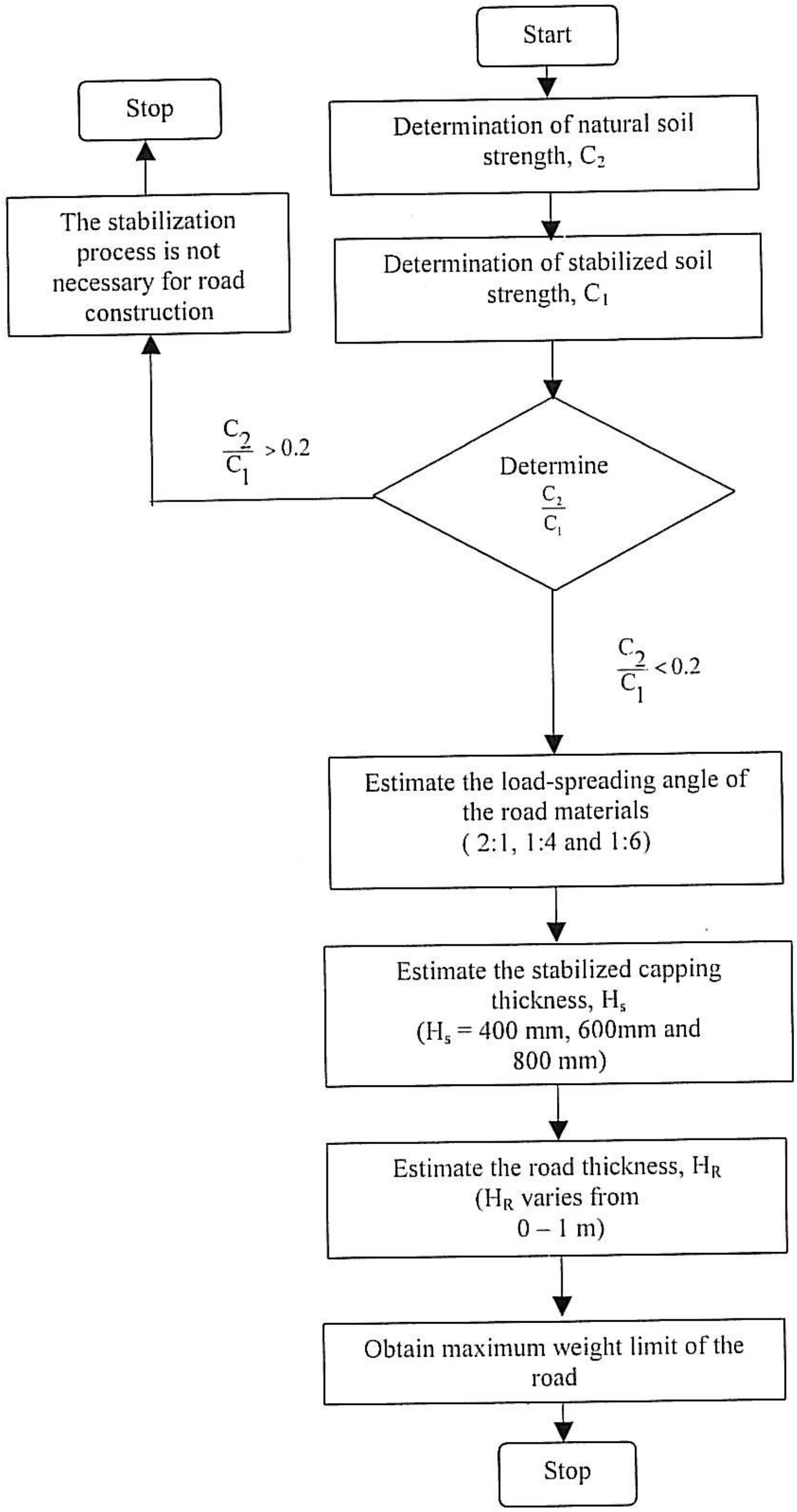


Figure 4.18: Flowchart to determine the weight limit of road constructed on lime-stabilized capping layer underlain by soft clay layer

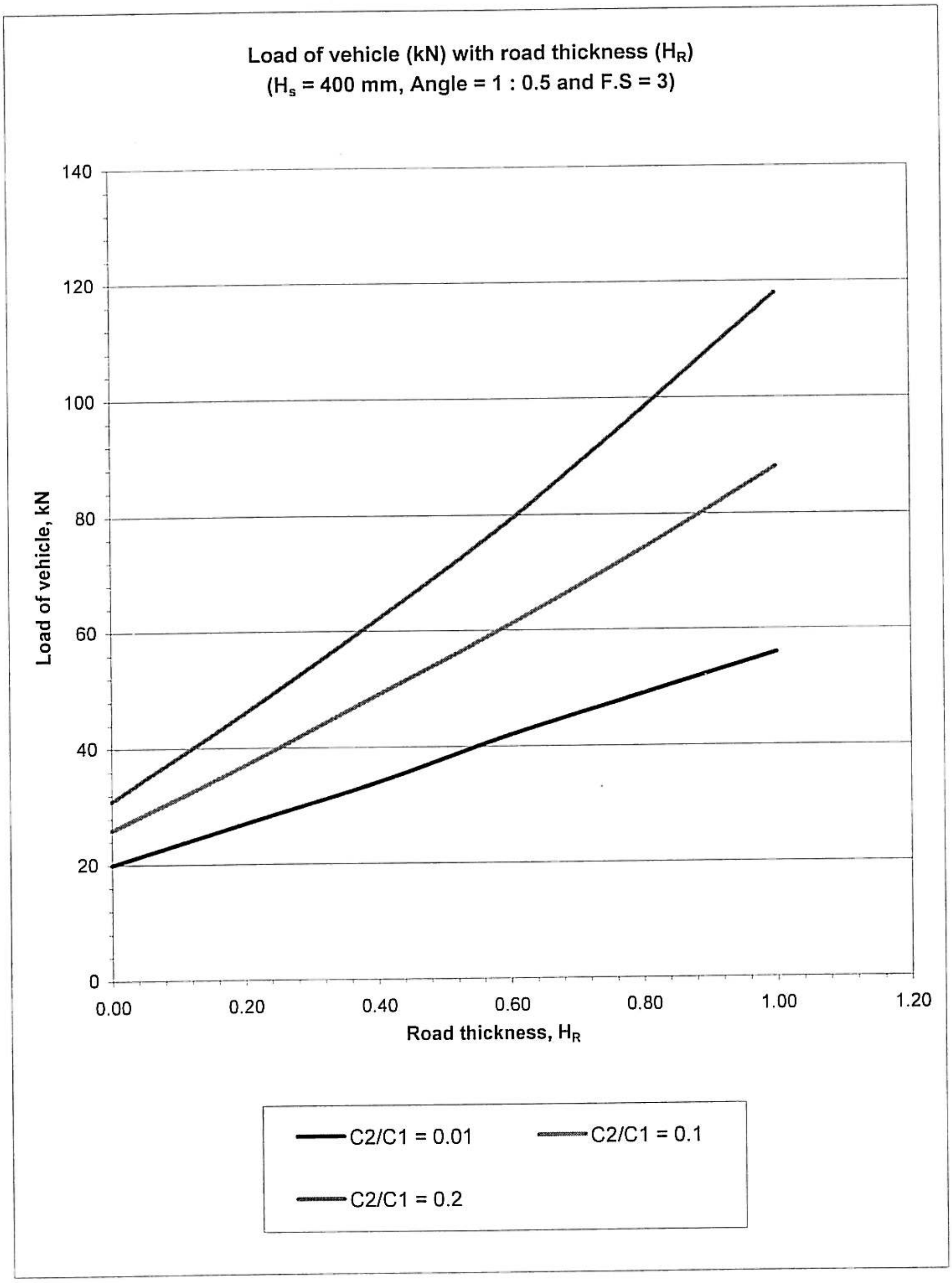


Figure 4.19: Chart No.1 (Hs = 400 mm, Angle = 1 : 0.5 and F.S = 3)

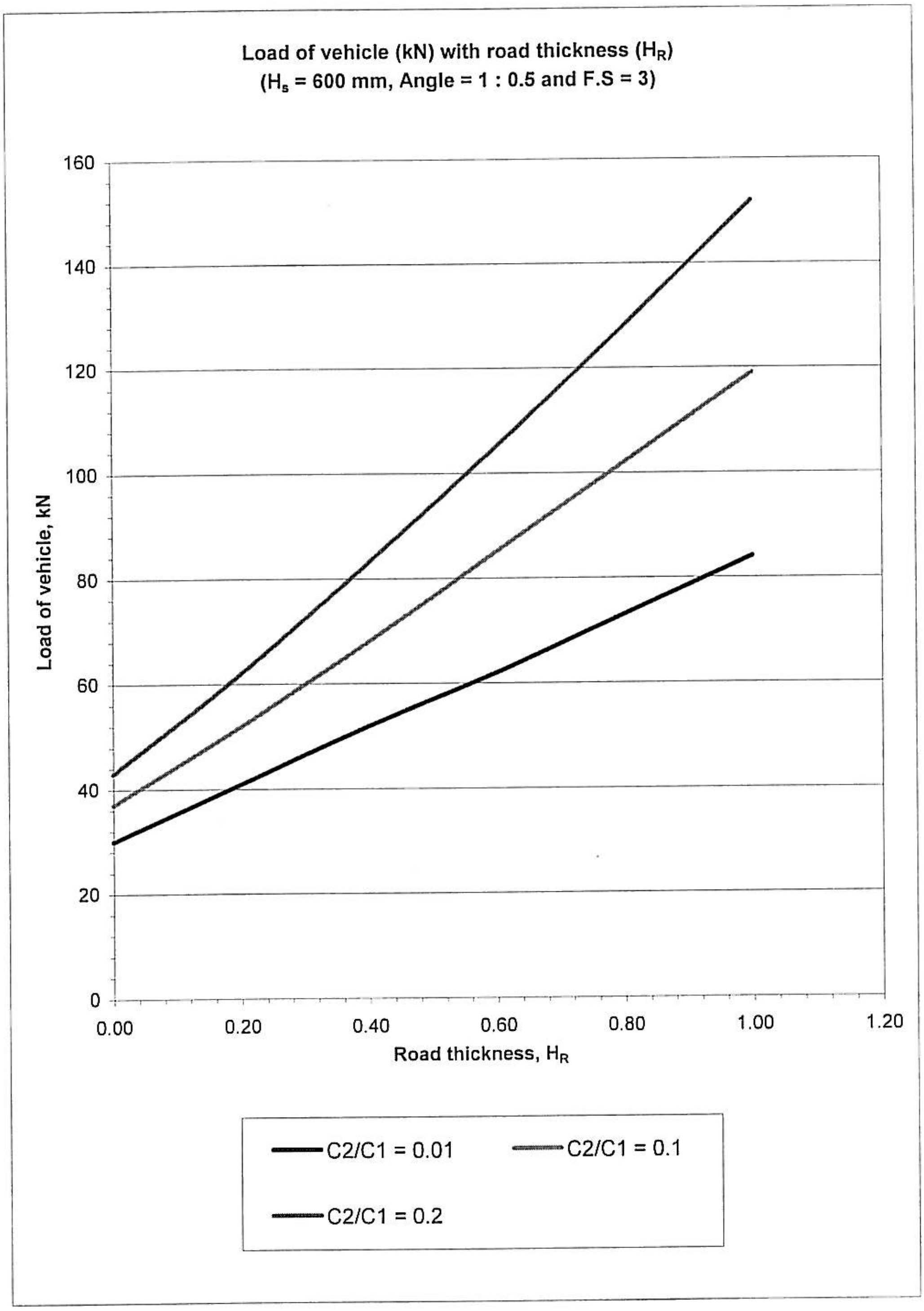


Figure 4.20: Chart No.2 (Hs = 600 mm, Angle = 1 : 0.5 and F.S = 3)

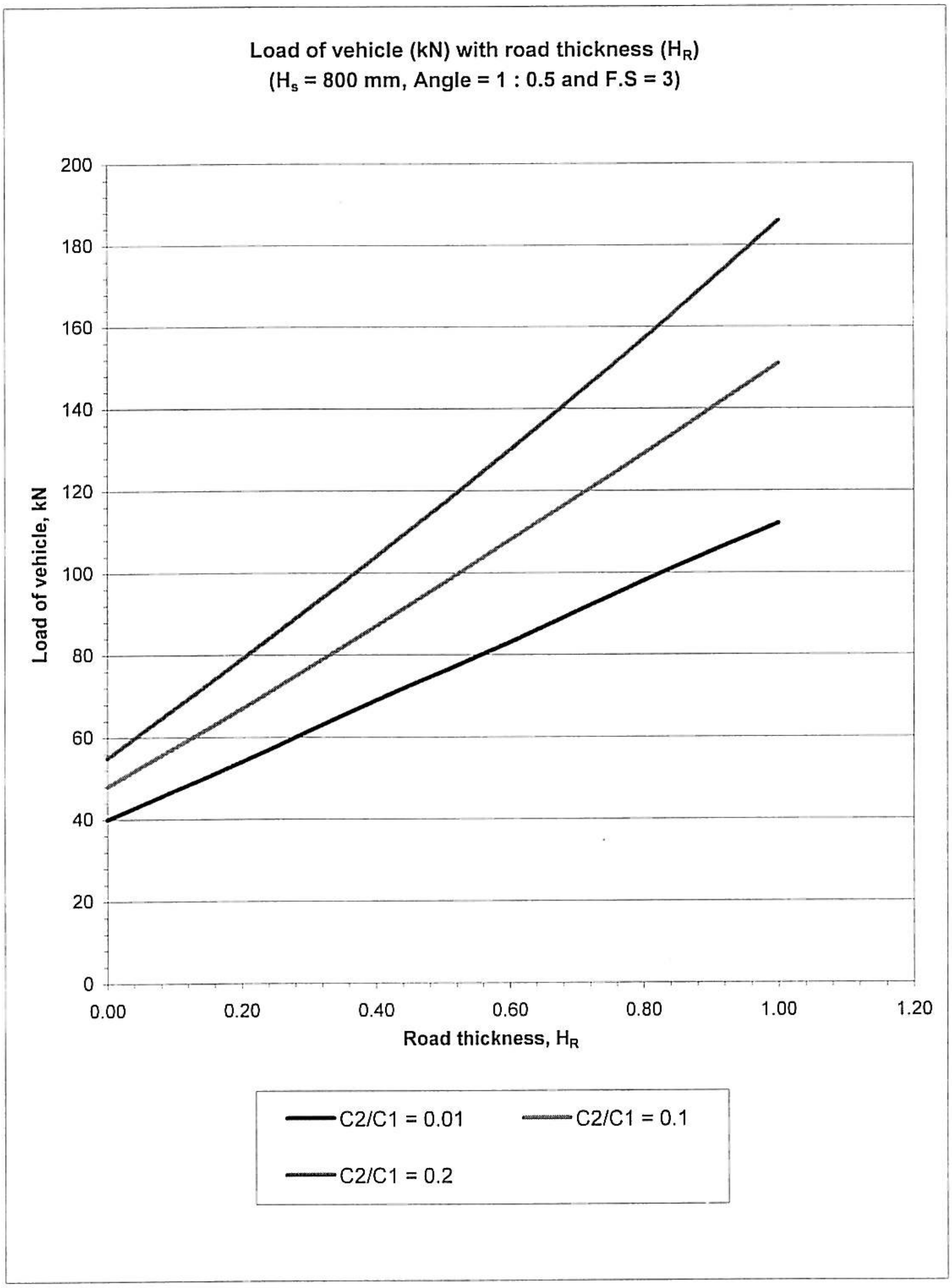


Figure 4.21: Chart No. 3 (Hs = 800 mm, Angle = 1 : 0.5 and F.S = 3)

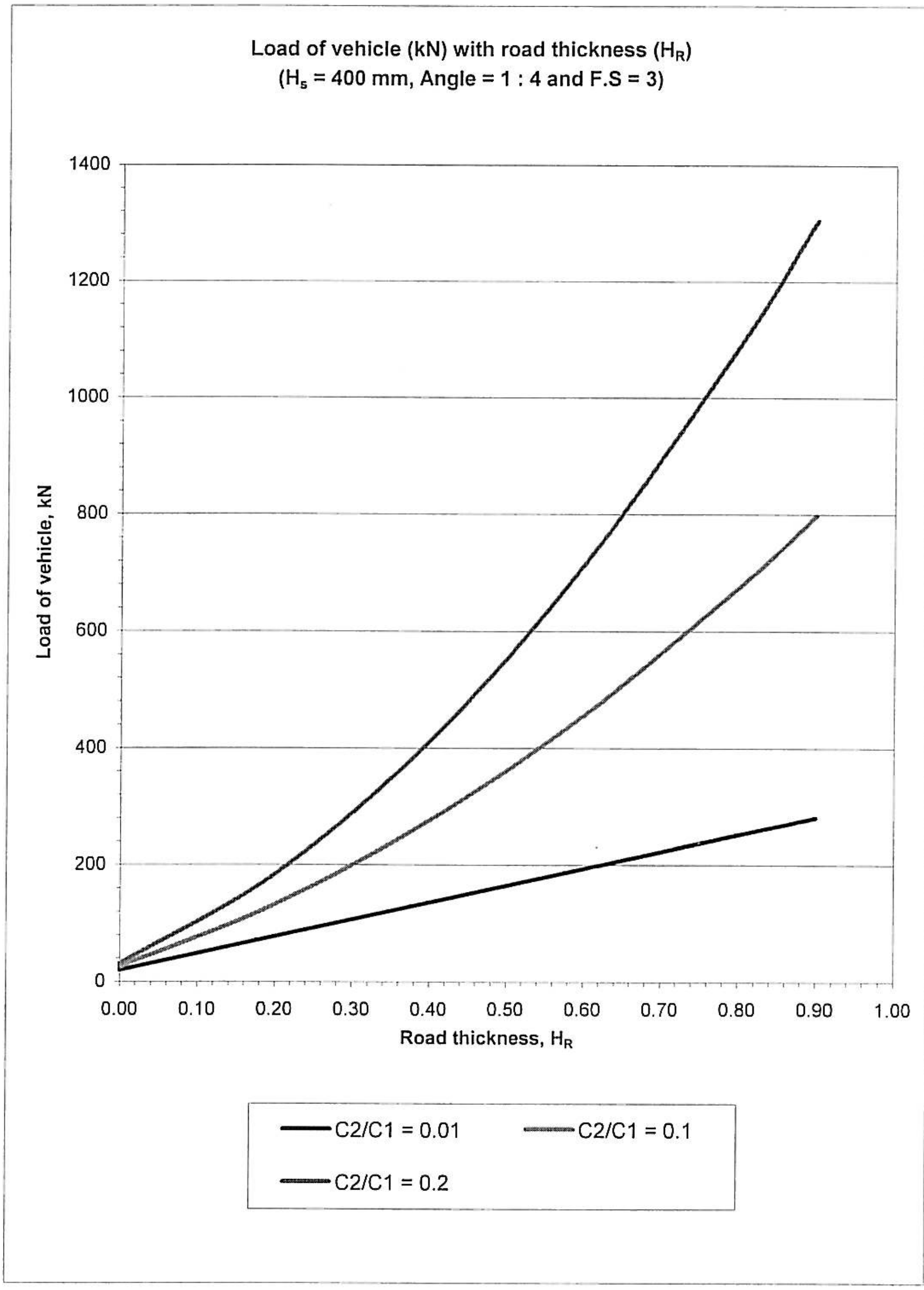


Figure 4.22: Chart No. 4 (Hs = 400 mm, Angle = 1 : 4 and F.S = 3)

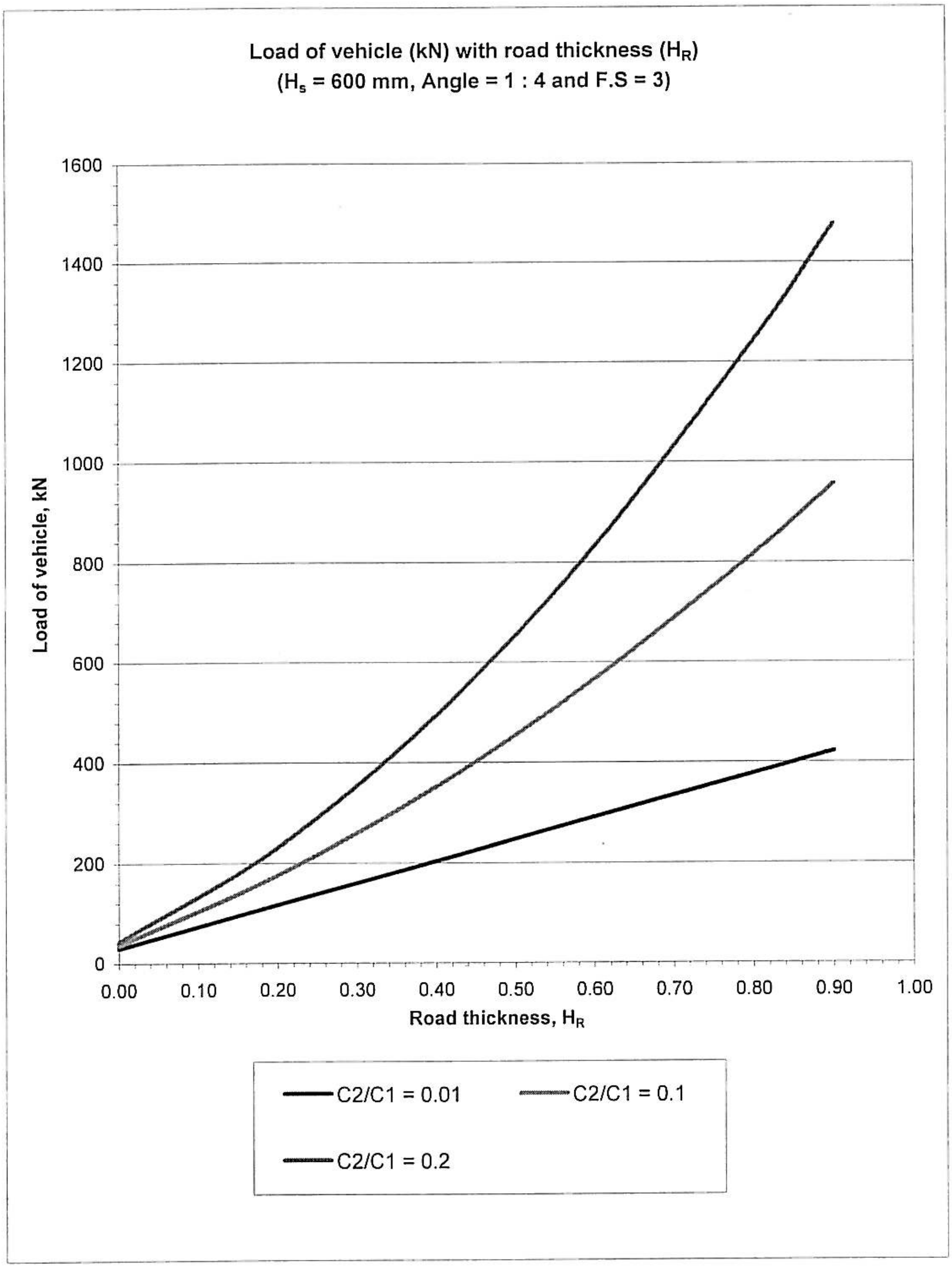


Figure 4.23: Chart No. 5 (Hs = 600 mm, Angle = 1 : 4 and F.S = 3)

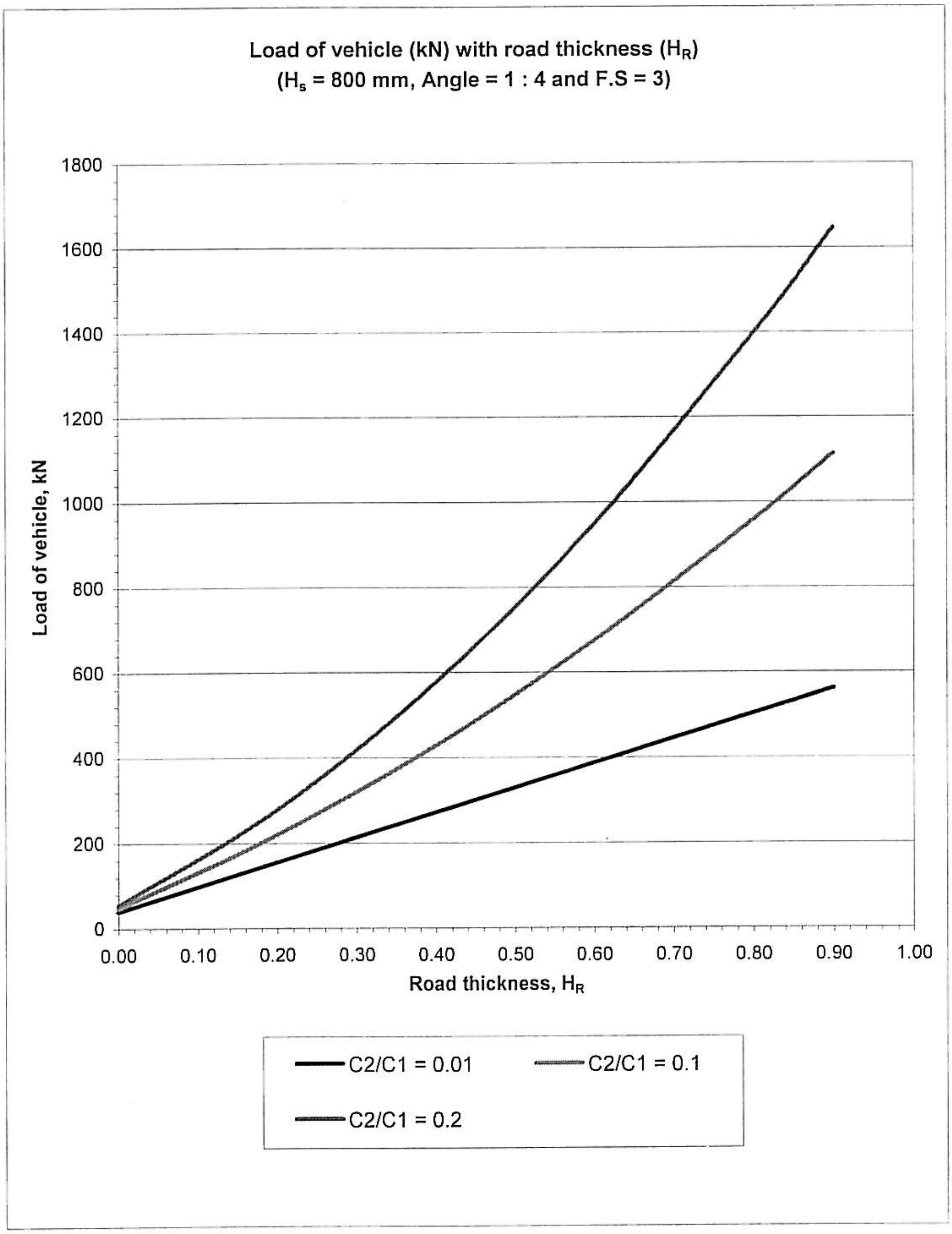


Figure 4.24: Chart No. 6 (Hs = 800 mm, Angle = 1 : 4 and F.S = 3)

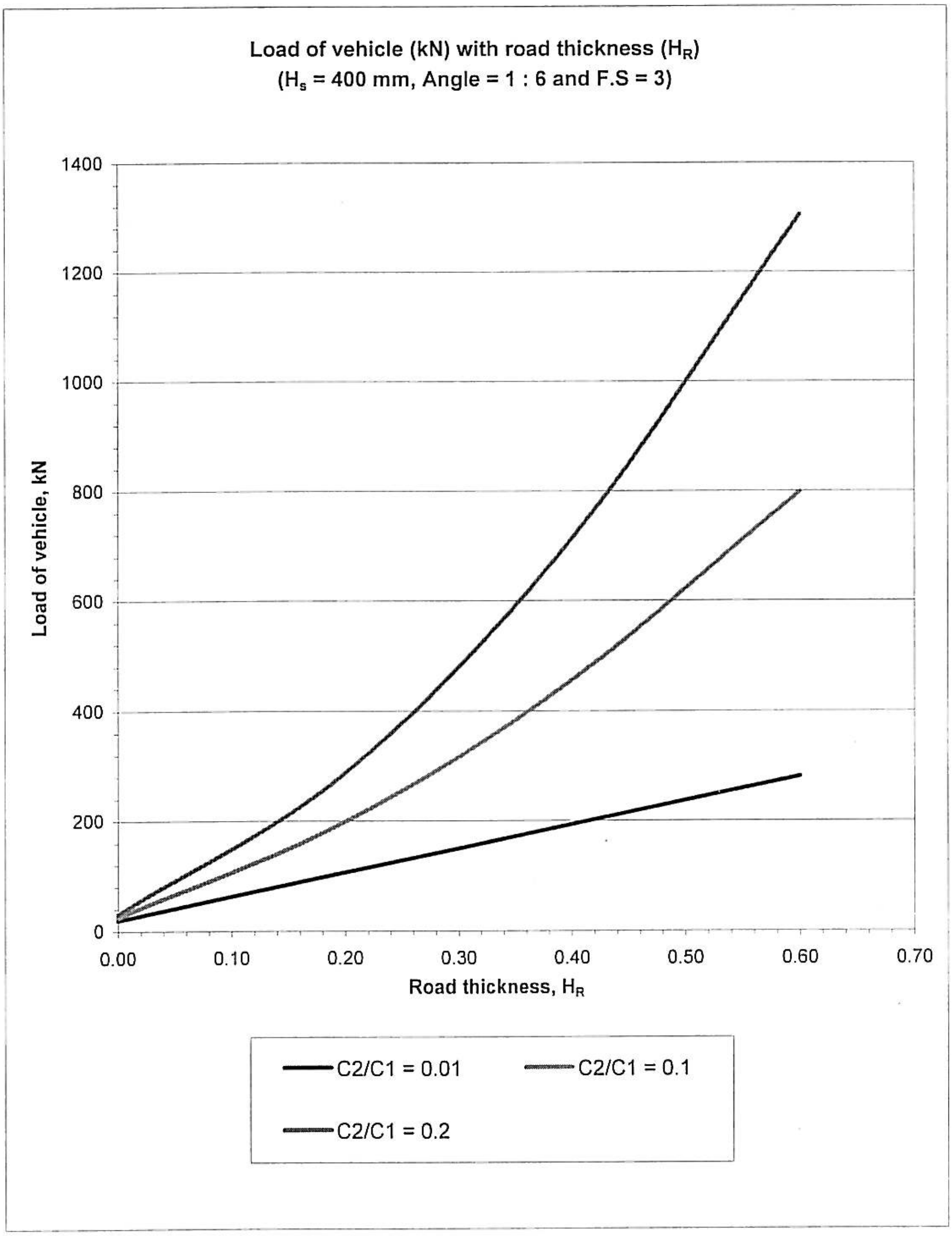


Figure 4.25: Chart No. 7 (Hs = 400 mm, Angle = 1 : 6 and F.S = 3)

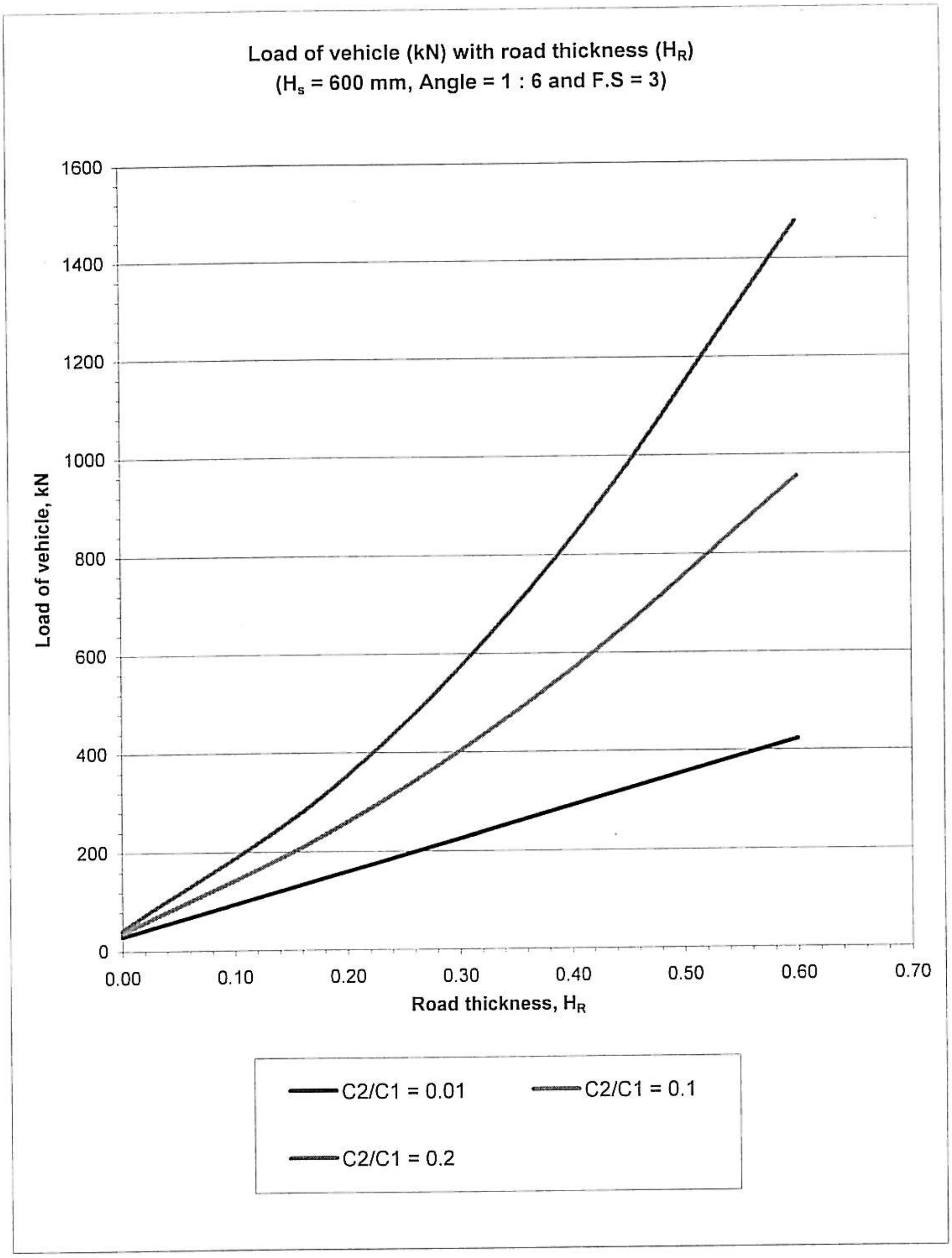


Figure 4.26: Chart No.8 (Hs = 600 mm, Angle = 1 : 6 and F.S = 3)

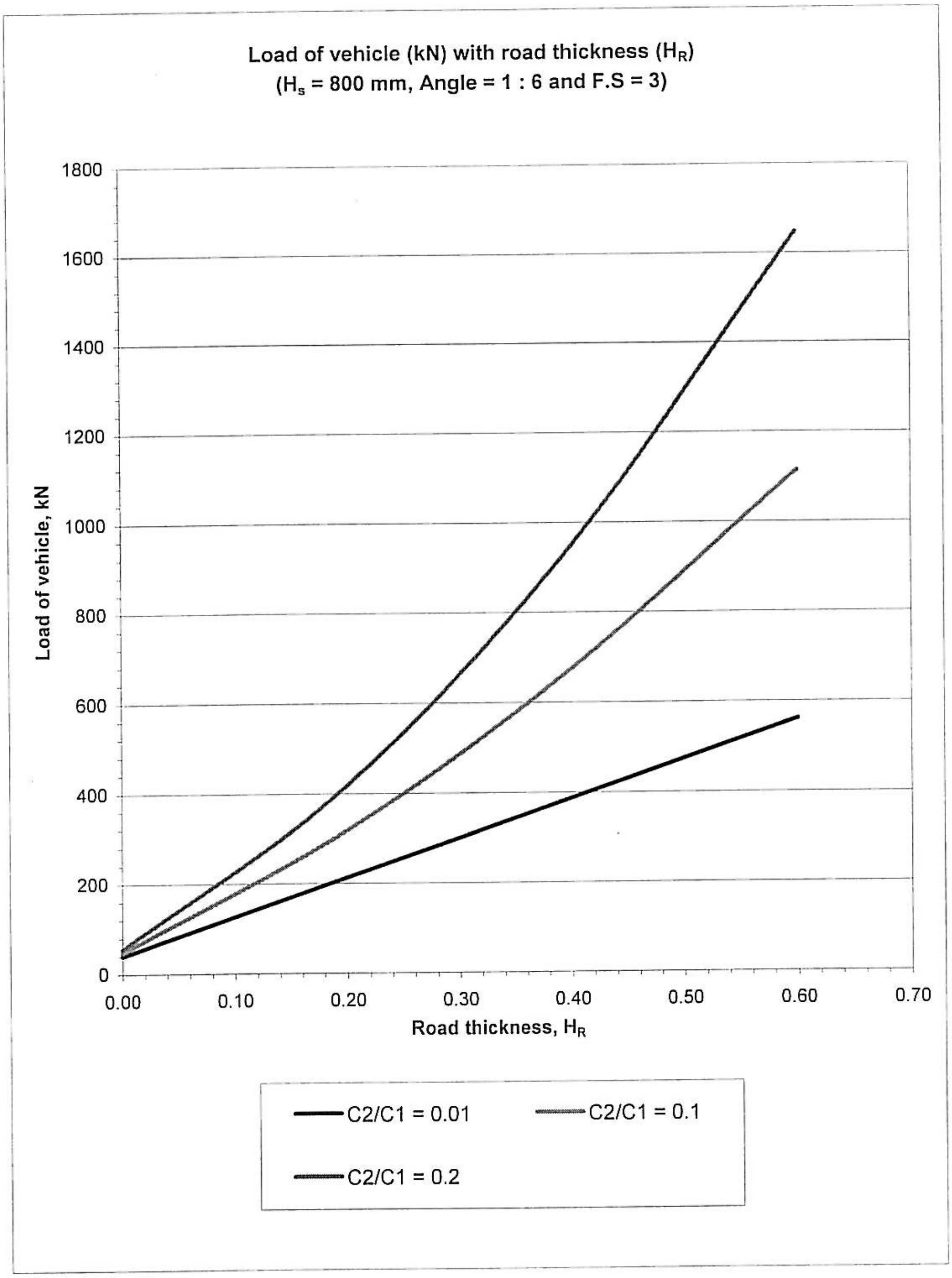


Figure 4.27: Chart No. 9 (Hs = 800 mm, Angle = 1 : 6 and F.S = 3)

From the design charts (Figure 4.19 to 4.27), it is shown that by increasing the thickness of the lime-stabilized capping layer, a corresponding increase in the capacity to sustain higher vehicular load is exhibited. This obviously indicates an increase in the bearing capacity of the structure. For stiff road materials having load-spreading angle of 1:4 and 1:6, the thickness of the road is less than 1 m.

4.7.6 Example of Design Calculation

An example of the weight limit design calculation is given as follows. This example is only for one point on the curve.

a. Measure the undrained shear strength of the soft layer, C_2

As an example, in this research, C_2 = 21.26 kN/m^2

b. Measure the undrained shear strength of the stabilized layer, C_1 at 28 days of curing period

$C_1 = 5.7622t + 51.303$ = $5.7622(28) + 51.303$

= 212.64 kN/m^2

c. Length of tyre imprint, L = 0.273 m

d. The maximum width of vehicle tyre, b = 0.275 m

f. Estimate the load-spreading angle of the road material = 1 : 4

g. Estimate road thickness, H_R = 0.6 m

h. Estimate stabilized thickness, Hs = 0.6 m

g. From Figure 4.17, calculate the effective width of the contact stress, B_{eff}

$$= b + 8H_R$$
$$= 0.275 + 8(0.6)$$
$$= 5.075 \text{ m}$$

h. Calculate the ultimate bearing capacity, q_u using Equations 2.4 and 2.5:

$H = H_s$ = 600 mm

= 0.6 m

$B = B_{eff}$ = 5.075 m

$\frac{C_2}{C_1}$ = 0.10

From Figure 4.28, $\frac{C_a}{C_1} = 0.72$

C_a = 0.72(212.64)

= 153.10

$$q_u = \left(1 + 0.2\frac{B}{L}\right)5.14C_2 + \left(1 + \frac{B}{L}\right)\left(\frac{2C_aH}{B}\right) + \gamma_1 D_f \leq q_t$$

$$q_u = \left(1 + 0.2\frac{5.075}{0.273}\right)5.14(21.26) + \left(1 + \frac{5.075}{0.273}\right)\left(\frac{2(155.10)(0.6)}{5.075}\right) \leq q_t$$

$$q_u = 1233.99\frac{kN}{m^2}$$

$$q_t = \left(1 + 0.2\frac{B}{L}\right)5.14C_1 + \gamma_1 D_f$$

$$q_t = \left(1 + 0.2\frac{5.075}{0.273}\right)5.14(212.64)$$

$q_t = 5156.73 \dfrac{kN}{m^2}$ $\qquad$ $q_u \leq q_t$

i. Factor of safety = 3

From Equation 4.8,

$$q_n = \frac{q_u}{3} \quad = \quad \frac{1233.99}{3} \quad = \quad 411.33 \frac{kN}{m^2}$$

From Equation 4.9, the maximum load can be calculated:

$$q_n = \frac{\text{Load(kN)}}{B_{eff} \times L}$$

$$\text{Load (kN)} = q_n \times [(B_{eff} \times L]$$

$$\text{Load (kN)} = 411.33 \times (5.075 \times 0.273)$$

$$\text{Load (kN)} = 570 \text{ kN}$$

$$= 57 \text{ tonne}$$

From the example, the road thickness of 0.6 m with load-spreading angle 1 : 4 which construct on the 600 mm thickness of lime-stabilized capping layer can sustain load of 57 tonne before the sub-grade being overstress. The load capacity will reduce if predetermined road thickness reduces. Therefore, the load capacity is influenced by the several factors includes road materials, road thickness, thickness of lime-stabilized capping layer and the undrained shear strength of the soft clay layer.

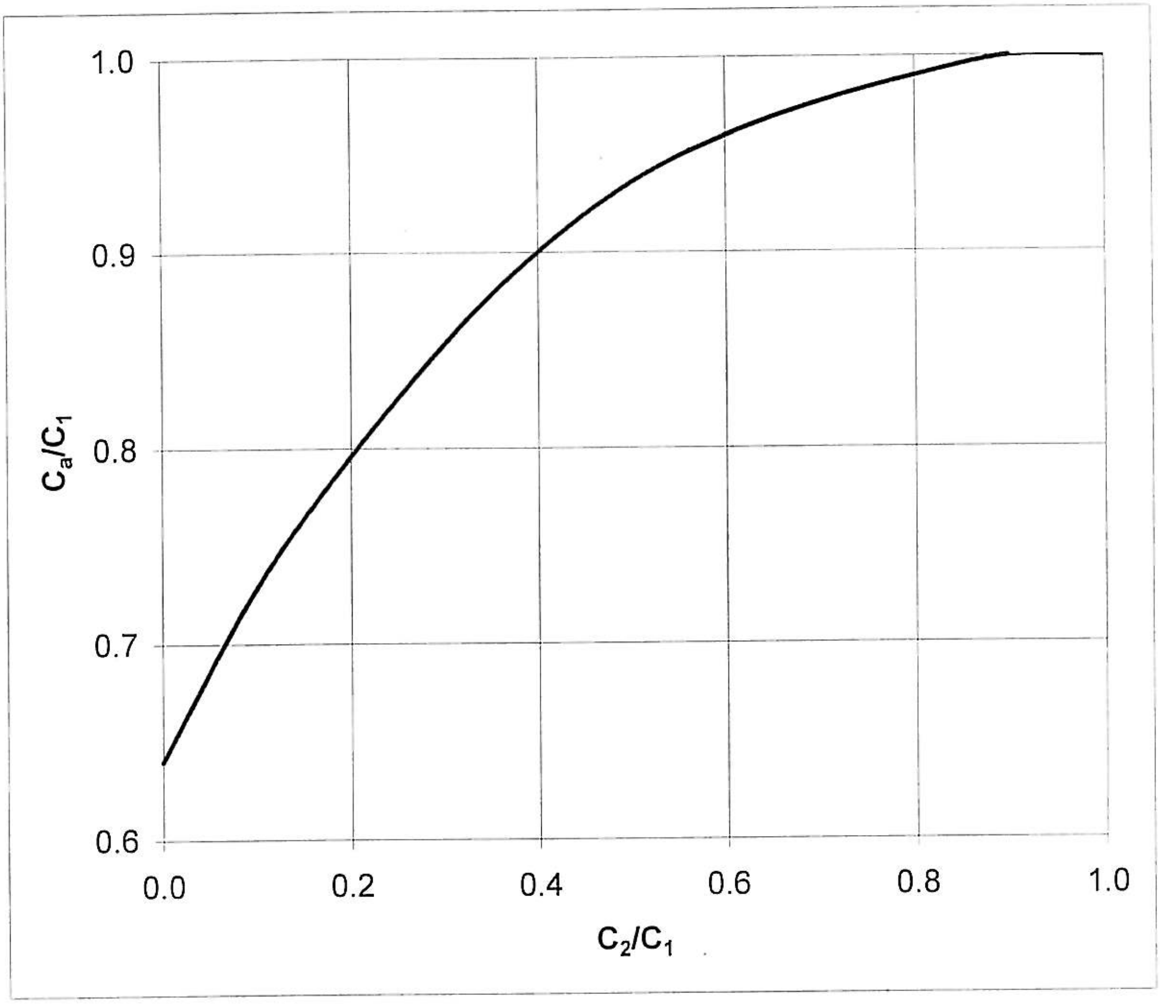


Figure 4.28: The value of $\frac{C_2}{C_1}$ (after Braja, 1999)

4.7.7 Road Thickness Design Using Manual On Pavement Design (Arahan Teknik Jalan (ATJ) 5/85)

According to this manual, highway design is conceptually based on two factors, namely, strength of sub-grade and traffic experienced by the road during its design life. In road thickness design, CBR values used range from 2 % to 15 %, and the Equivalent Standard Axle (ESA) values range from 1 x 10^4 to 1 x 10^8. By following the thickness design nomograph procedure (Figure 4.29), the corrected equivalent thickness, ($T_{A'}$)

value is obtained and based on this $T_{A'}$ value, the thicknesses of the road layers are determined using Equation 4.10. This section will discuss what we can achieve if this manual is used in the road design with weak sub-grades. From the design CBR of 2 %, several values of the ESA and $T_{A'}$ are obtained as shown in Table 4.26. The road thickness for each case is determined from Equation 4.10.

$$T_{A'} = a_1 D_1 + a_2 D_2 + \ldots + a_n D_n \tag{4.10}$$

where,

a_1, a_2 ... a_n are the structural coefficients of each layer as shown in Table 4.27

D_1, D_2, ... D_n are the thickness of each layer

As an example, the structural coefficients obtained from Table 4.27 are used to determine the road thickness. The coefficient values are 1.0 for asphalt concrete, 0.32 for base course and 0.23 for sub-base. By using Equation 4.10, the thickness of the road is determined. As a guideline, the minimum thickness of each layer is based on the value from Table 4.28. The calculated thickness of the road for each case is shown in Table 4.29. The design calculations are as follows.

Table 4.26: ESA and structural number ($T_{A'}$) obtained from the design CBR value of 2%

Equivalent Standard Axle (ESA)	Structural Number ($T_{A'}$), cm
1×10^4	11.20
5×10^4	13.50
1×10^5	16.40
5×10^5	21.50
1×10^6	24.40
5×10^6	32.10
5×10^7	38.40
1×10^8	No value

Consider,

CBR = 2 %, ESA = 1 x 10^6 and from Figure 5.13, $T^{A'}$ = 24.40 cm

From Equation 4.10,

$$T_{A'} = a_1 D_1 + a_2 D_2 + \ldots + a_n D_n$$

Assign $D_1 = 13$

$D_2 = 14$

$D_3 = 30$

$$T_{A'} = 1 \text{ x } 13 + 0.32 \text{ x } 14 + 0.23 \text{ x } 30$$

$$= 24.40$$

Taking into consideration the minimum thickness requirements, the pavement structure comprises of the following thicknesses.

Wearing	-	5 cm
Binder	-	8 cm
Base	-	14 cm
Sub-base	-	30 cm

Therefore, the total thickness of the road, which includes the sub-base, road base, binder course and wearing course is 57 cm.

Table 4.27: Structural layer coefficient (after Manual on Pavement Design, 1985)

Component	Type of layer	Property	Coefficient
Wearing and Binder Course	Asphalt concrete		1.00
Base Course	Dense Bituminous Macadam	Type 1: Stability > 400 kg	0.80
		Type 2: Stability > 300 kg	0.55
	Cement Stabilized	Unconfined Compressive Strength (7 days) 30 – 40 kg/cm2	0.45
	Mechanically Stabilized Crushed Aggregate	CBR 80 %	0.32
Sub-base	Sand, laterite etc.	CBR 20 %	0.23
	Crushed Aggregate	CBR 30 %	0.25
	Cement Stabilized	CBR 60%	0.28

Table 4.28: Minimum layer thickness (after Manual on Pavement Design, 1985)

Type of layer		Minimum thickness (cm)
Wearing Course		4
Binder Course		5
Base Course	Bituminous	5
	Wet Mix	10
	Cement Treated	10
Sub-base Course	Granular	10
	Cement Treated	15

Table 4.29: Road thickness for design CBR of 2%

Equivalent Standard Axle (ESA)	Structural Number ($T_{A'}$) cm	Road Thickness (cm)				Total road thickness, H_R (cm)
		Wearing course	Binder course	Road base	Sub-base	
1×10^4	11.20	-	5.7	10	10	25.70
5×10^4	13.50	-	5	13	19	37.00
1×10^5	16.40	-	5	17	26	48.00
5×10^5	21.50	4	5	19	28	56.00
1×10^6	24.40	5	8	14	30	57.00
5×10^6	32.10	5	10	28	35	78.00
5×10^7	No value	-	-	-	-	-
1×10^8	No value	-	-	-	-	-

As has been noted previously, the lime-stabilized capping layer underlain the soft clay layer can provide the foundation for road construction. The mean CBR value of this structure (CBR_m) can be determined using Equation 4.11 as proposed by Manual on Pavement Design (Arahan Teknik Jalan). This equation is suitable for the case of varying CBR value within 1 m depth of the sub-grade, especially when soil stabilization has been undertaken.

$$CBR_m = \left[\frac{h_1 CBR_1^{\frac{1}{3}} + h_2 CBR_2^{\frac{1}{3}} + \ldots + h_n CBR_n^{\frac{1}{3}}}{1000} \right]^3 \tag{4.11}$$

Based on the proposed road thickness design using lime-stabilized layer underlain soft clay layer, the thickness of lime-stabilized soils and clay layer are considered is as shown in Table 4.30. The undrained shear strength of the lime-stabilized layer and the soft clay layer at 28 days of curing period is 212.64 kN/m^2 and

23 kN/m^2 respectively. The CBR value for each layer can be obtained based on their undrained shear strengths using Equation 4.5. From this equation, the CBR value of the lime-stabilized layer is 10.21 %, and for the soft clay, it is 1.10 %. From the thickness and CBR value of these layers, the design CBR value of the lime-clay system is calculated using Equation 4.11. These design CBR values of the system for varying thicknesses of the lime stabilized capping layer and the soft clay layer are tabulated in Table 4.30. It is evident from the tabulated values that the design CBR value of the system increases with increasing thickness of the lime-stabilized capping layer.

Table 4.30: Thickness of the various layers in relation to the CBR of the system

Thickness of stabilized layer (mm)	Thickness of soft clay layer (mm)	CBR_m (%)
400	600	3.29
600	400	5.04
800	200	7.32

This finding obviously facilitates road design. It may contribute to the reduction of the road thickness and increasing the ESA capacity of the road. Therefore, the proposed road thickness design based on this concept of lime-stabilized sub-grade can serve as a supplementary method in the normal practice of highway engineering. The road thickness obtained from the lime-stabilized capping layer is shown in Tables 4.31, 4.32 and 4.33, and the example calculation of the road thickness reduction is given as follows.

CBR value of lime-stabilized soil	=	10.21 %
CBR value of soft clay	=	1.10 %
Thickness of lime-stabilized layer	=	600 mm
Thickness of soft clay	=	400 mm

From Equation 4.11, CBR for design,

$$CBR = \left[\frac{h_1 CBR_1^{\frac{1}{3}} + h_2 CBR_2^{\frac{1}{3}} + \ldots + h_n CBR_n^{\frac{1}{3}}}{1000}\right]^3$$

$$CBR = \left[\frac{600 \text{ x } (10.21)^{\frac{1}{3}} + 400 \text{ x } (1.10)^{\frac{1}{3}}}{1000}\right]^3$$

$$CBR = 5.04\ \%$$

From the previous example calculation, the road thickness for the CBR of 2 % and ESA of 1 x 10^6, the total of road thickness was 57 cm with 13 cm of the asphaltic concrete, 14 cm of the base-course and 30 cm of the sub-base layer. When considering the design CBR of lime-clay system with the same value of ESA, the road thickness was determined as follows:

CBR = 5.04 %, ESA = 1 x 10^6 and from Figure 5.13, $T_{A'}$ = 20.80 cm

From Equation 5.3,

$$T_{A'} = a_1 D_1 + a_2 D_2 + \ldots + a_n D_n$$

Assign $D_1 = 13$

$D_2 = 10$

$D_3 = 20$

$$T_{A'} = 1 \text{ x } 13 + 0.32 \text{ x } 10 + 0.23 \text{ x } 20$$
$$= 20.80$$

Taking into consideration the minimum thickness requirements, the pavement structure comprises of the following thickness:

Wearing	-	5 cm
Binder	-	8 cm
Base	-	10 cm
Sub-base	-	20 cm

The total road thickness is reduced from 57 cm to 43 cm because the value of the design CBR has increased from 2 % to 5.04 %, maintaining the same thickness of the asphaltic concrete and ESA value. This finding can obviously reduce the cost of road construction besides minimizing the use of granular materials for the base, sub-base layer, and as replacement for the weak sub-grade.

Table 4.31: Road thickness for lime-stabilized sub-grade (CBR = 3.29%)

Equivalent Standard Axle (ESA)	Structural Number ($T_{A'}$) Cm	Road Thickness (cm)				Total road thickness, H_R (cm)
		Wearing course	Binder course	Road base	Sub-base	
1×10^4	No value	-	-	-	-	-
5×10^4	12.00	-	5	11	15	31.00
1×10^5	14.50	-	5	15	18	38.00
5×10^5	19.80	4	5	18	22	49.00
1×10^6	22.50	5	8	15	20	48.00
5×10^6	28.00	5	10	22	26	63.00
5×10^7	39.50	8	10	38	41	97.00
1×10^8	43.50	10	10	41	45	106.00

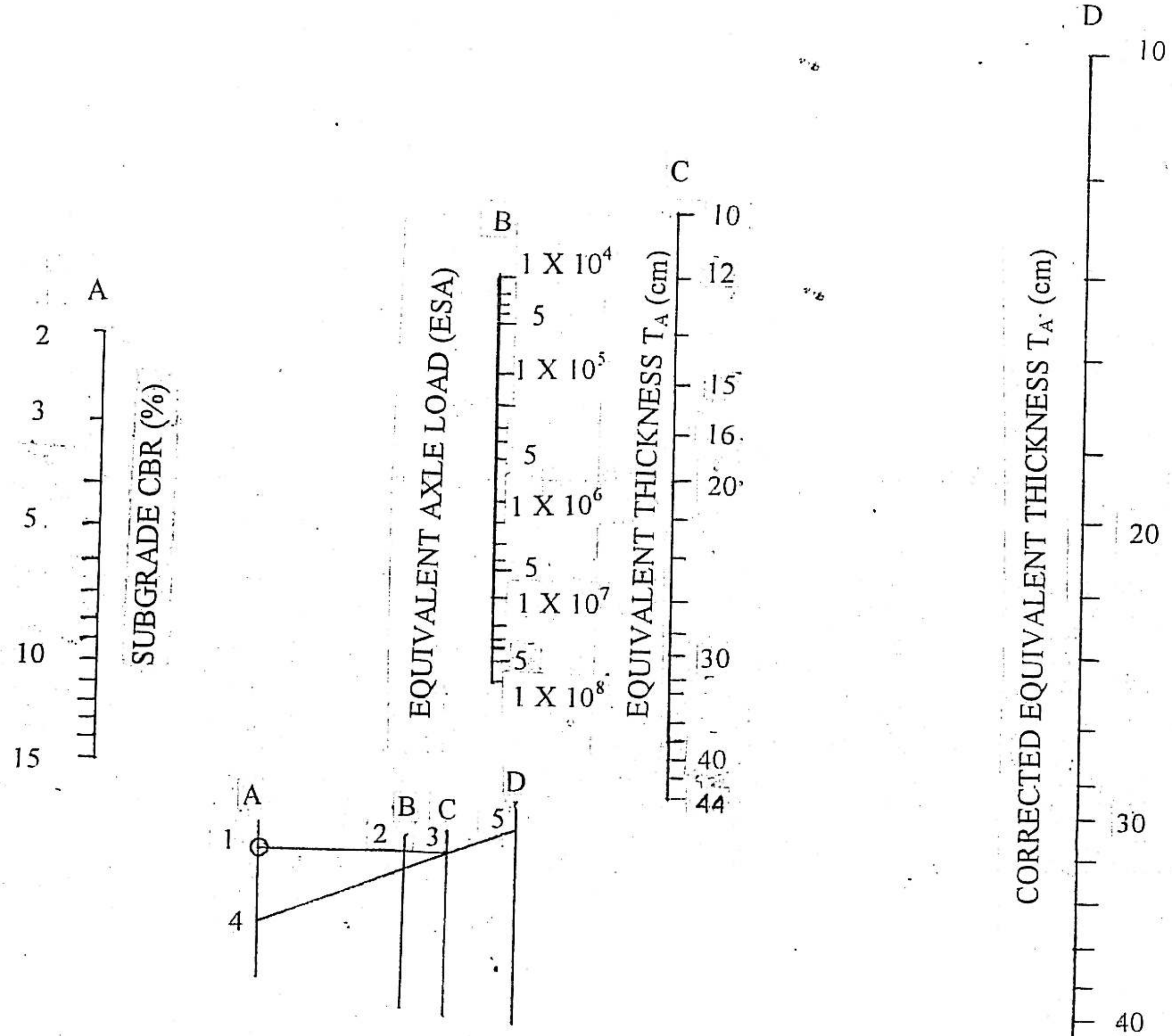
Table 4.32: Road thickness for lime-stabilized sub-grade (CBR = 5.04%)

Equivalent Standard Axle (ESA)	Structural Number ($T_{A'}$) Cm	Road Thickness (cm)				Total road thickness, H_R (cm)
		Wearing course	Binder course	Road base	Sub-base	
1 x 10^4	No value	-	-	-	-	-
5 x 10^4	10.80	-	5	8	14	27.00
1 x 10^5	13.20	-	5	11	20	13.20
5 x 10^5	18.00	4	5	14	20	43.00
1 x 10^6	20.80	5	8	10	20	43.00
5 x 10^6	25.80	5	10	15	26	56.00
5 x 10^7	36.00	8	10	28	39	85.00
1 x 10^8	38.50	10	10	29	40	89.00

Table 4.33: Road thickness for lime-stabilized sub-grade (CBR = 7.32%)

Equivalent Standard Axle (ESA)	Structural Number ($T_{A'}$) Cm	Road Thickness (cm)				Total road thickness, H_R (cm)
		Wearing course	Binder course	Road base	Sub-base	
1 x 10^4	No value	-	-	-	-	-
5 x 10^4	No value	-	-	-	-	-
1 x 10^5	12.00	-	5	11	15	29.00
5 x 10^5	16.50	4	5	12	16	37.00
1 x 10^6	19.00	5	8	10	12	35.00
5 x 10^6	24.00	5	10	10	25	50.00
5 x 10^7	33.20	8	10	25	31	74.00
1 x 10^8	36.00	10	10	25	35	80.00

From the tables, it is clear that the lime-stabilized capping layer has reduced the thickness of the road and increased the capacity to sustain heavier traffic loads. As an example, for the design CBR of 2%, is was not possible to construct the road with an ESA of 1×10^8. However, when the lime-stabilized method was employed, the road was successfully designed to sustain an ESA of 1×10^8.



1. CBR = 3
2. ESA
3. T_A for CBR = 3
4. Design CBR
5. Required T_A

Figure 5.13: Thickness design nomograph (after Manual on Pavement Design, 1985)

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

Classification tests were conducted on soil sample from Tg. Pelepas, Universiti Teknologi Malaysia and Kulai to investigate the suitability of the materials for the stabilization process. Chemical tests conducted on hydrated lime to confirm the suitability and percentage of lime for mix design of soil-lime mixture. The UCS and CBR test on the unstabilized soil gave the correlation between the soil strength and the moisture content. Correlation was also established between the shear strength and CBR value. Physical model testing simulated the construction of lime-stabilized sub-grade as a capping layer on soft clay layer to facilitate the design process which was based on geotechnical aspect. The performance of this structure in terms of settlements were observed and bearing capacity values were measured. From the bearing capacity values, the weight limit design with consideration not to overstress the sub-grade was developed. This chapter summarizes all the highlighted results and draw conclusion that could benefit the engineers and other researchers working with lime stabilization as capping layer for road construction.

5.1.1 Soil-lime Mixture Design

a. From the soil properties testing, the suitability of soil for lime stabilization was established. The cohesive soil with organic content less than 2 %, Plasticity Index (PI) more than 10 % and Clay Fraction (CF) of more than 10 % represented the suitable soil for lime stabilization method.

b. Basically, the amount of water in the soil-lime mixture was based on the 95 % of Maximum Dry Density (MDD). In this study, the amount of water used for simulating the soft soil condition was from the moisture content based on the minimum CBR value of 2 %.

c. Empirical correlations between Vane Shear strength with Unconfined Compressive Strength (UCS) test and Unconfined Compressive Strength (UCS) with California Bearing Ratio (CBR) test was established.

d. The amount of lime for modification was based on the Initial Consumption of Lime (ICL) test. From the ICL test result, the amount of lime of 3 % is obtained. However, for stabilization process, more lime is required and it is determined from strength test. Based on CBR tests on stabilized samples with 5 % of lime, the increase in CBR value is more than 100 % after curing period of 28 days.

e. The strength development of the lime-stabilized soil is depends on the curing period. Initially, the strength of soil-lime mixture increases. However for the first two weeks, the strength development is not consistent. It may due to the modification process which not yet produce permanent bonding. After 14 days, the strength increases with the curing period.

f. The lime stabilized soft clay for road construction is capable of providing a strong layer for capping. From the deformation model result, the lime-stabilized capping layer reduced the settlement to more than 100 %.

g. The bearing capacity of the lime-stabilized capping layer underlain by soft clay layer is dependent on the thickness of the lime-stabilized capping layer and the curing period.

5.1.2 Weight Limit Design for Road With Lime-Stabilized Capping Layer

a. From the proposed weight limit design chart, the maximum allowable load can be obtained at a point before the ultimate bearing capacity of the sub-grade is exceeded.

b. The maximum allowable load obtained which does not overstressing the sub-grade can be used as an important criterion to control the life span of roads.

c. This method has presented an additional precaution in road design.

5.2 Recommendations

a. In this research only curing period was considered during investigation of lime stabilization strength development. Therefore, further research should be carried out with incorporating the temperature effect on the lime-stabilized soil.

b. In this study, road materials are assumed to be a homogenous material. Further research should be carried out with multi-layer system which represent the actual road condition.

c. The physical model was tested under static load condition. Hence, dynamic load should be studied to obtain a more comprehensive result as dynamic loading portraits the actual condition of roads.

d. The bearing capacity model testing was conducted at unsoaked condition. It is suggest for further research which conduct the test with the soaked lime-stabilized capping layer to look the effect of the water infiltration to the structure.

e. The lime-stabilization method is recommended for the construction of road with low traffic intensity like in housing estates and motorcycle lanes. This is because, lime-stabilized soil underlain soft clay layer can sustain load of 2 to 5 tonne without having any road thickness. Therefore it is suitable for motorcycle lanes.

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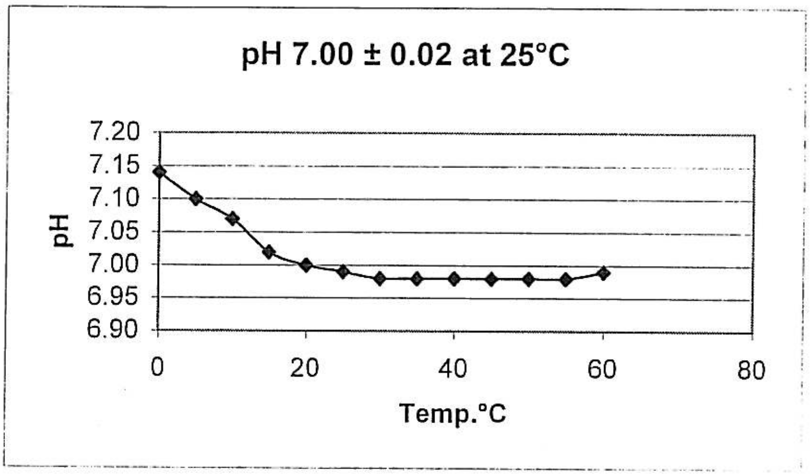
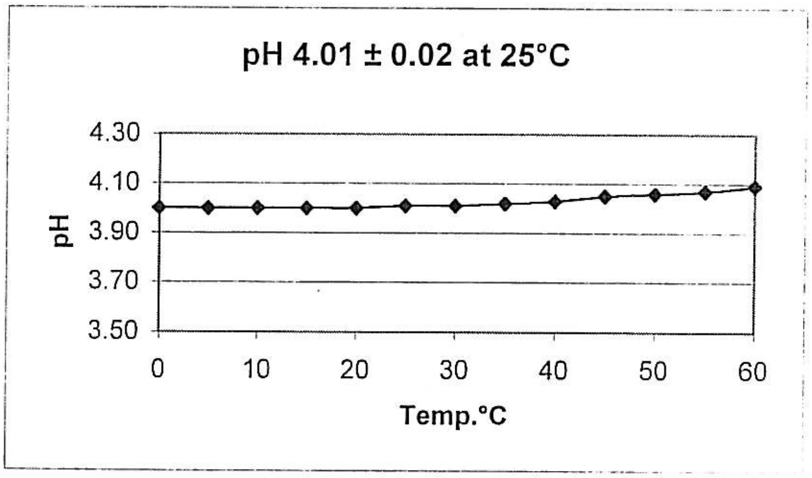
APPENDIX A

Calibration data for pH metre

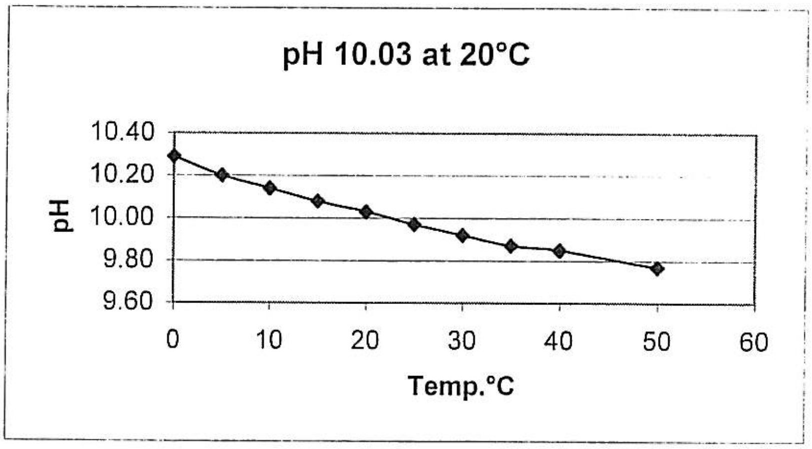
Temp.°C	pH
0	7.14
5	7.10
10	7.07
15	7.02
20	7.00
25	6.99
30	6.98
35	6.98
40	6.98
45	6.98
50	6.98
55	6.98
60	6.99



Temp.°C	pH
0	4.00
5	4.00
10	4.00
15	4.00
20	4.00
25	4.01
30	4.01
35	4.02
40	4.03
45	4.05
50	4.06
55	4.07
60	4.09



Temp.°C	pH
0	10.29
5	10.20
10	10.14
15	10.08
20	10.03
25	9.97
30	9.92
35	9.87
40	9.85
50	9.77

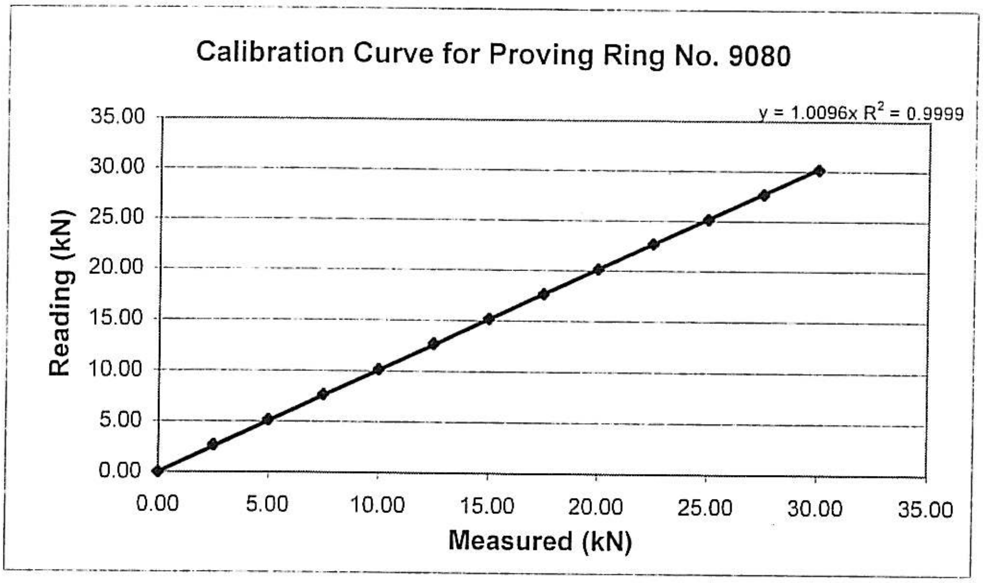


APPENDIX B

Calibration data for proving ring (No. 9080)

Apparatus : Proving Ring
Serial No. : 9080 (30 kN)

No.	Measured (kN)	Reading (kN) 1	2	3	Means Reading (kN)
1	0.00	0.00	0.00	0.00	0.00
2	2.50	2.58	2.70	2.70	2.66
3	5.00	5.08	5.22	5.22	5.17
4	7.50	7.53	7.74	7.76	7.68
5	10.00	10.08	10.26	10.23	10.19
6	12.50	12.60	12.84	12.74	12.74
7	15.00	15.07	15.30	15.18	15.18
8	17.50	17.59	17.75	17.75	17.70
9	20.00	20.11	20.25	20.18	20.18
10	22.50	-	22.72	22.70	22.71
11	25.00	-	25.20	25.22	25.21
12	27.50	-	27.72	27.72	27.72
13	30.00	-	30.19	30.19	30.19

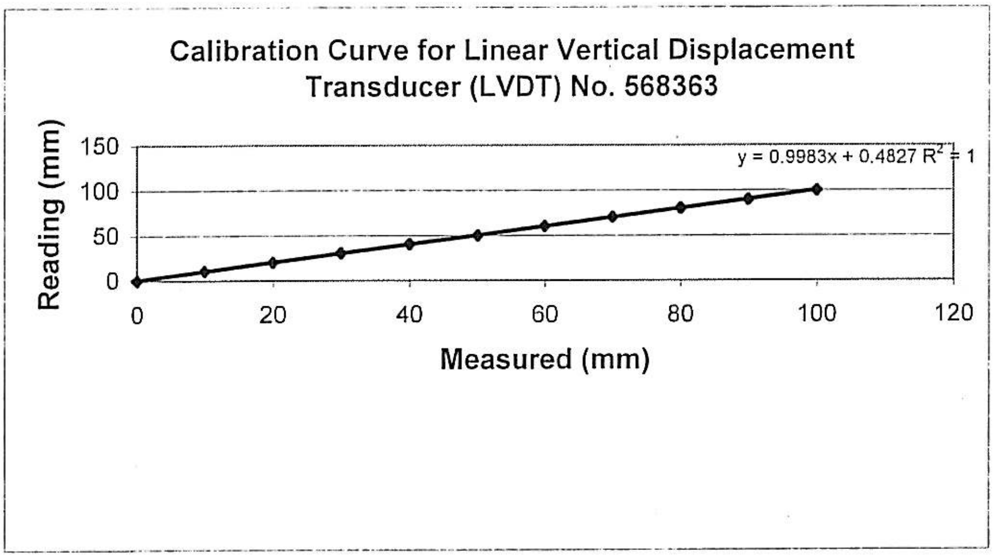


APPENDIX C

Calibration data for Linear Vertical Displacement Transducer (LVDT), Load cell and CBR Machine

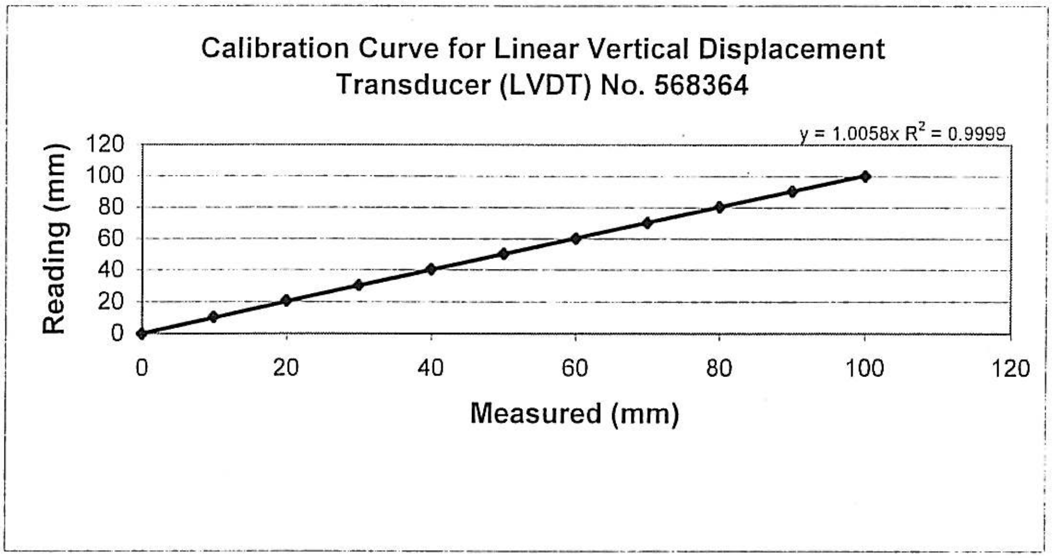
Apparatus : Linear Vertical Displacement Transducer (LVDT)
Serial No. : 568363

No.	Measured (mm)	Reading (kN)			Means Reading (mm)
		1	2	3	
0	0	0.00	0.00	0.00	0.00
1	10	10.57	10.39	10.85	10.60
2	20	20.34	20.38	20.71	20.48
3	30	30.51	30.58	30.77	30.62
4	40	40.13	41.19	40.97	40.76
5	50	50.10	50.33	50.37	50.27
6	60	60.47	60.46	60.56	60.50
7	70	70.47	70.26	70.43	70.39
8	80	80.35	80.48	80.68	80.50
9	90	89.71	90.31	90.28	90.10
10	100	100.28	100.29	100.16	100.16



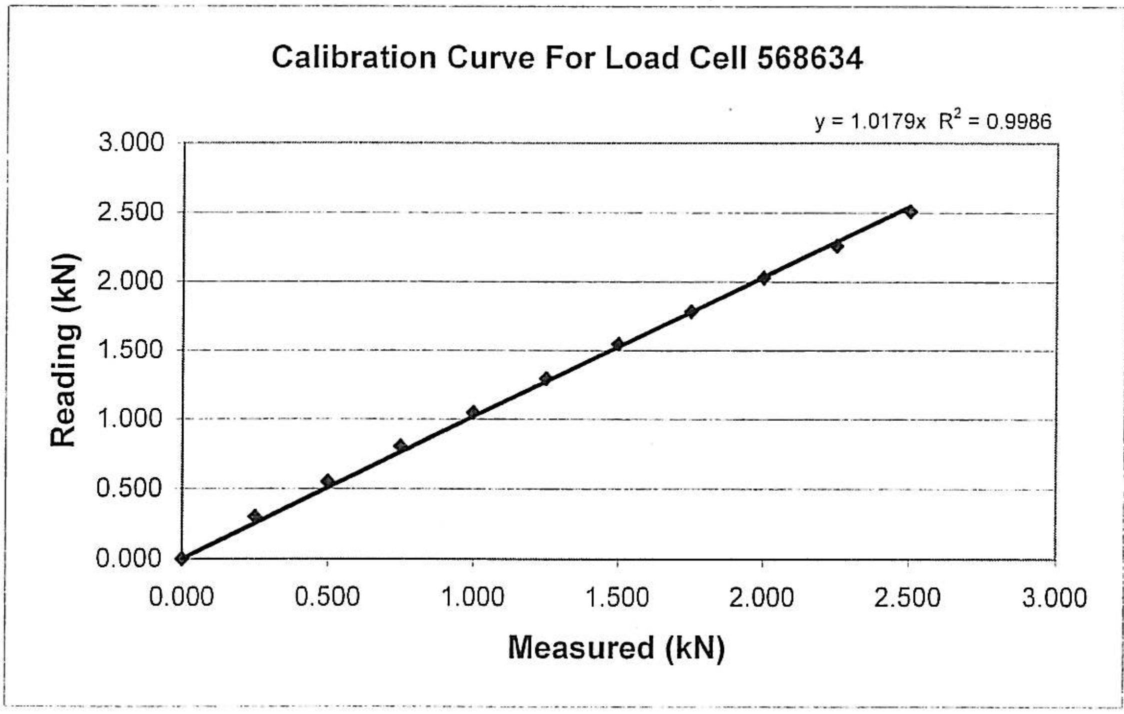
Apparatus : Linear Vertical Displacement Transducer (LVDT)
Serial No. : 568364

No.	Measured (mm)	Reading (kN)			Means Reading (mm)
		1	2	3	
0	0	0.00	0.00	0.00	0.00
1	10	10.49	10.68	10.51	10.56
2	20	20.70	20.81	20.54	20.68
3	30	30.44	30.64	30.55	30.54
4	40	40.72	40.52	40.36	40.53
5	50	50.49	50.45	50.25	50.40
6	60	60.43	60.34	60.15	60.31
7	70	70.56	70.32	70.24	70.37
8	80	80.61	80.51	80.49	80.54
9	90	90.29	90.68	90.67	90.55
10	100	100.16	100.08	100.08	100.11



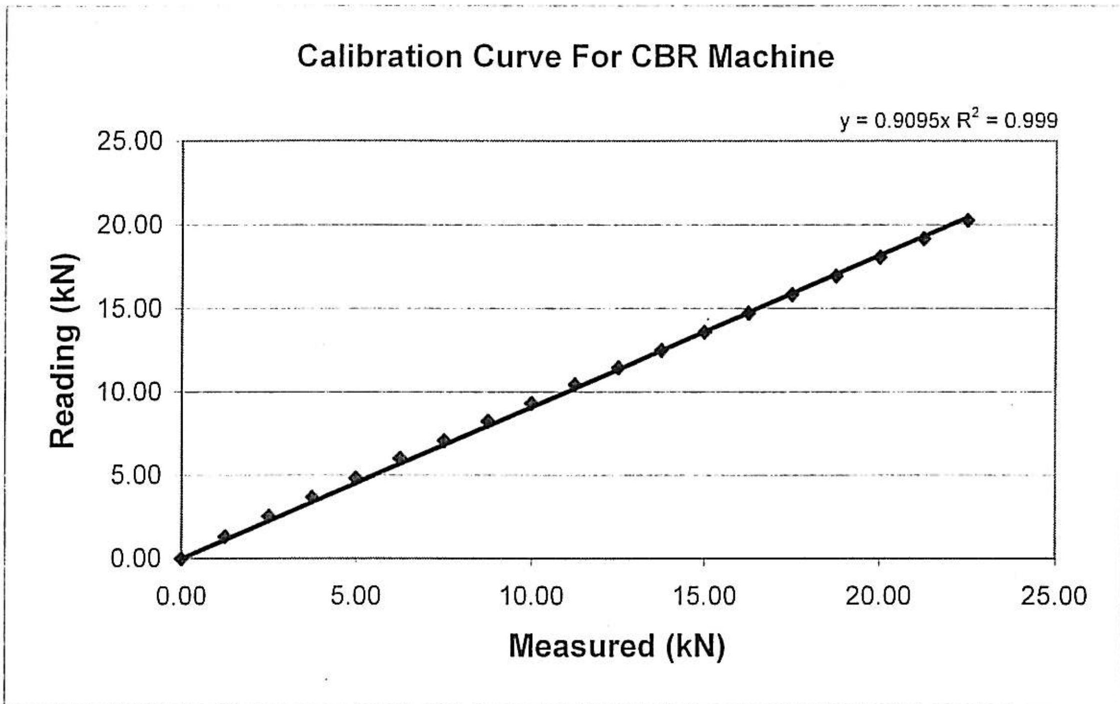
Apparatus : Load cell
Serial No. : 568364

No.	Measured	Reading (kN)			Means Reading
	(mm)	1	2	3	(mm)
0	0.000	0.000	0.000	0.000	0.000
1	0.250	0.296	0.303	0.309	0.303
2	0.500	0.552	0.554	0.549	0.552
3	0.750	0.812	0.804	0.798	0.805
4	1.000	1.045	1.047	1.052	1.048
5	1.250	1.316	1.274	1.292	1.294
6	1.500	1.558	1.548	1.539	1.548
7	1.750	1.781	1.799	1.778	1.786
8	2.000	2.027	2.053	2.014	2.031
9	2.250	2.274	2.255	2.255	2.261
10	2.500	2.519	2.513	2.498	2.510



Apparatus : ;BR Machine
Serial No. :

No.	Measured (mm)	Reading (kN) 1	Reading (kN) 2	Reading (kN) 3	Means Reading (mm)
1	0.00	0.00	0.00	0.00	0.00
2	1.25	1.30	1.50	1.20	1.33
3	2.50	2.50	2.80	2.40	2.57
4	3.75	3.60	3.90	3.60	3.70
5	5.00	4.70	5.00	4.70	4.80
6	6.25	5.90	6.20	5.90	6.00
7	7.50	7.00	7.20	7.00	7.07
8	8.75	8.20	8.40	8.10	8.23
9	10.00	9.30	9.50	9.20	9.33
10	11.25	10.40	10.60	10.30	10.43
11	12.50	11.50	11.60	11.30	11.47
12	13.75	12.50	12.60	12.40	12.50
13	15.00	13.60	13.70	13.50	13.60
14	16.25	14.70	14.90	14.60	14.73
15	17.50	15.80	16.00	15.70	15.83
16	18.75	16.90	17.10	16.80	16.93
17	20.00	18.10	18.20	17.90	18.07
18	21.25	19.20	19.30	19.00	19.17
19	22.50	20.20	20.40	20.20	20.27

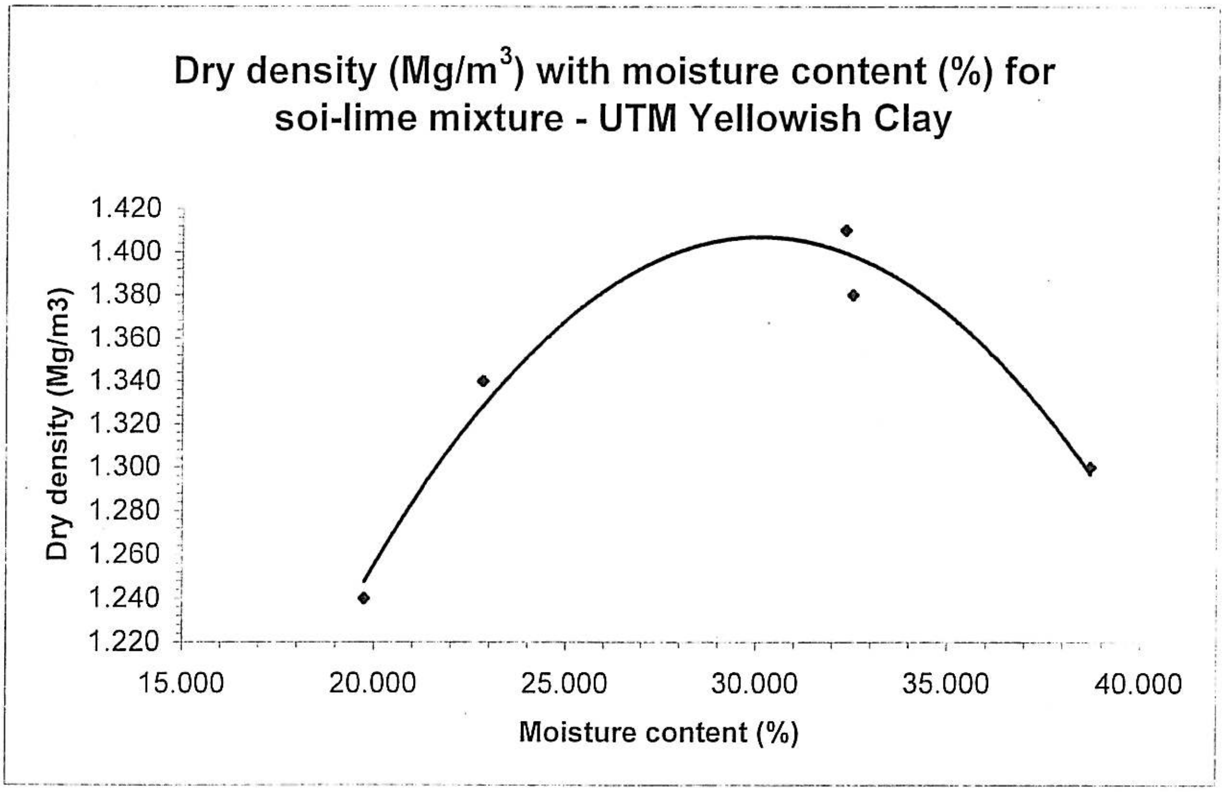


APPENDIX D

Result of compaction test for UTM Yellowish Clay

Laboratory	Mak. Penyediaan JGP	Rammer used	2.5	kg
Operator	Mustafa	Mould used	1	L
Material	UTM Yellowish White Clay	Stabilizer	Hydrated lime	
Job	Vot 71382	Stabilizer content	5	%
Site	UTM			

Test Portion No.		1	2	3	4	5
Mass of mould + base + specimen (m2)	g	4.750	4.890	5.110	5.070	5.020
Mass of mould + base (m1)	g	3.260	3.240	3.250	3.240	3.240
Mass of compacted specimen (m2 - m1)	g	1.490	1.650	1.860	1.860	1.800
Bulk density	Mg/m^3	1.490	1.650	1.870	1.870	1.810
Moisture content (w)	%	19.760	22.840	32.350	32.540	38.720
Dry density	Mg/m^3	1.240	1.340	1.410	1.380	1.300



APPENDIX E

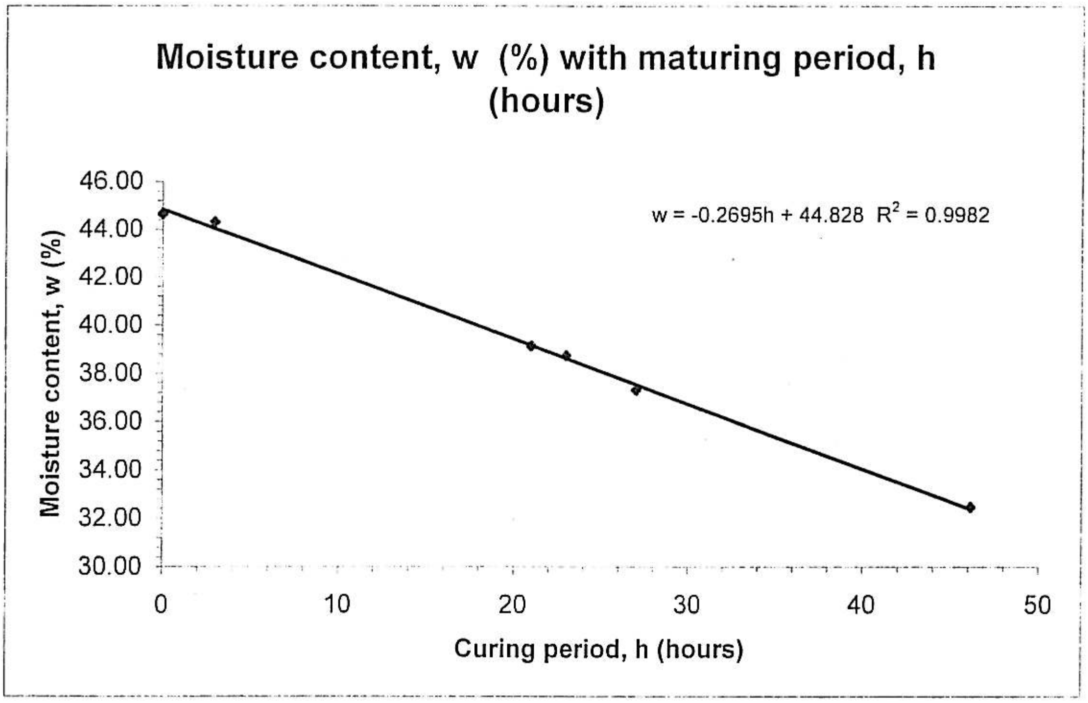
Correlation between moisture content with maturing period for UTM Yellowish Clay

Consitency of Work

Determination of maturing period for soil-lime mixture according to 95% Maximum Dry Density (MDD) of moisture content

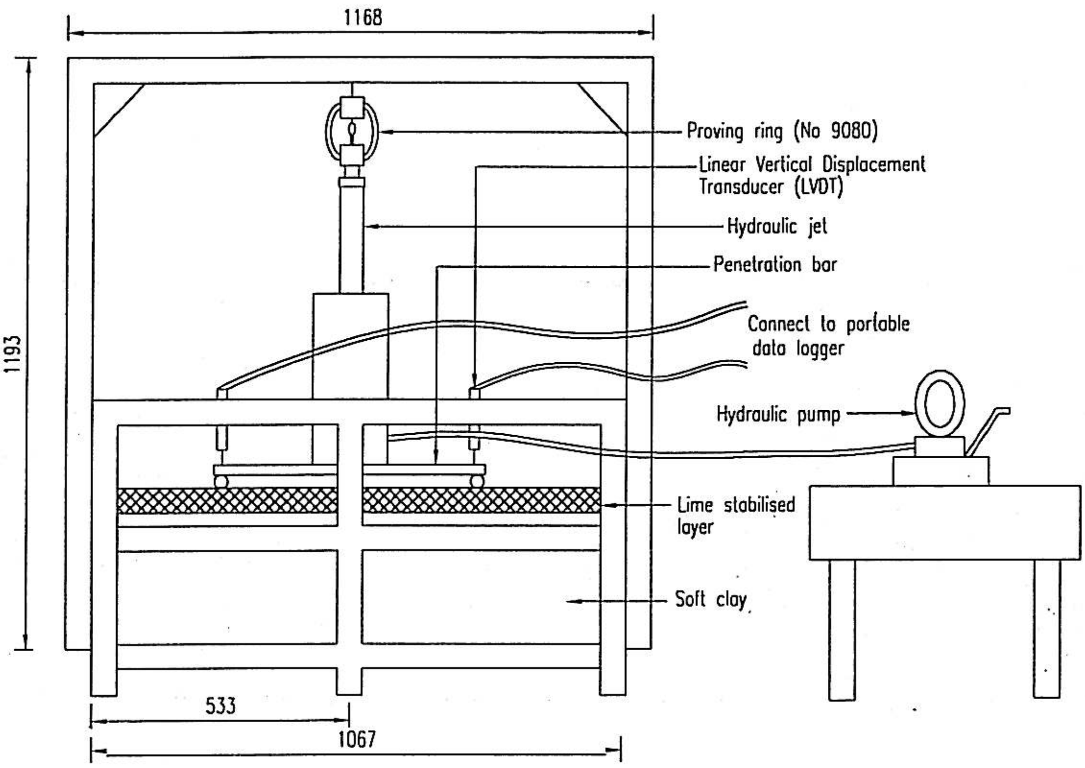
Date start	20/12/2002				Soil	UTM Yellowish Clay	
Time	9.15 a.m					600	gm
					Water	48	%
					Lime	5	%

Hours (curing period)		0	3	21	23	27	46.15
Container No.		107A	80A	70A	105A	32A	117A
Mass of container	m_1	9.69	9.60	9.32	9.77	9.44	9.56
Mass of container + soil	m_2	28.26	51.36	26.10	24.49	26.99	25.92
Mass of container + dry soil	m_3	22.53	38.54	21.38	20.38	22.22	21.91
Mass of water (m_2 - m_3)	m_4	5.73	12.82	4.72	4.11	4.77	4.01
Mass of dry soil (m_3 - m_1)		12.84	28.94	12.06	10.61	12.78	12.35
Moisturen content (w)	%	44.63	44.30	39.14	38.74	37.32	32.47



APPENDIX F

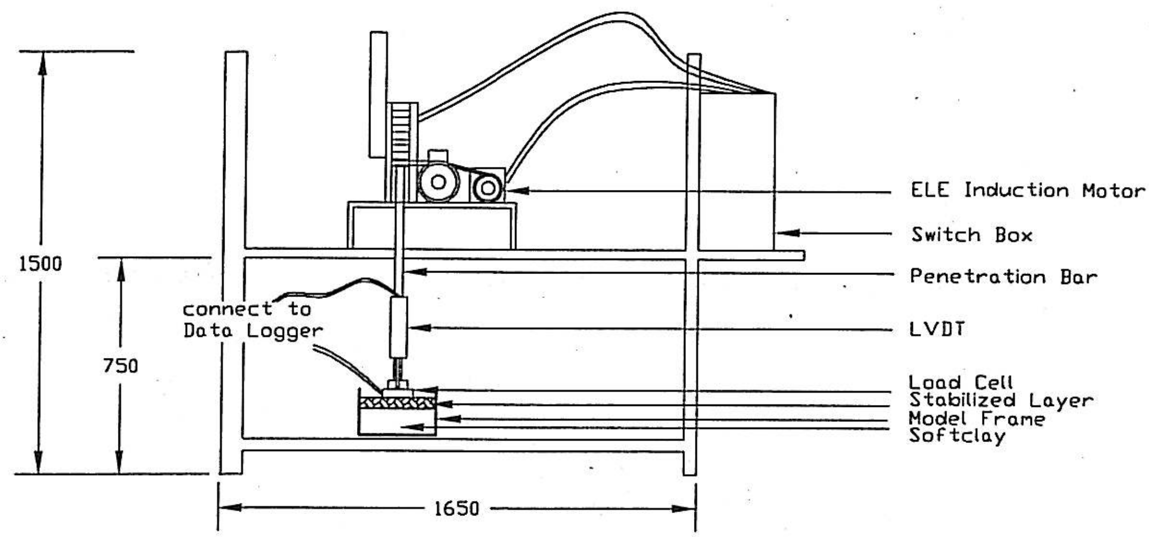
Schematic diagram for deformation model



All dimensions are in milimetres (mm)
All drawings are not to scale

APPENDIX G

Schematic diagram for bearing capacity model



All dimension are in milimeter (mm).
All drawing are not to scale.

APPENDIX H

Result of specific gravity

Determination of Particle Density

Location	Tg. Pelepas			Job ref.	Vot 71382	
				Boer hole/ Pit no.		
Soil description	PLP			Sample No		1
				Depth		
Test method	BS 1377: Part 2: 1990, Clause 8.3			Date		8/8/00
Method pf preparation	Small pyknometer method					
Small/large pyknometer						
Specimen reference			1	2	3	4
Pyknmeter number			1587	1591	1900	1576
Mass of bottle + soil + water	m_3	g	84.093	86.905	82.768	83.842
Mass of bottle + soil	m_2	g	35.724	38.805	37.288	36.210
Mass of bottle full of water	m_4	g	80.857	83.496	80.099	79.339
Mass of bottle	m_1	g	30.492	33.409	31.335	30.754
Mass of soil	$m_2 - m_1$	g	5.232	5.396	5.953	5.456
Mass of water in full bottle	$m_4 - m_1$	g	50.365	50.087	48.764	48.585
Mass of water used	$m_3 - m_2$	g	48.369	48.100	45.480	47.632
Mass of water equivalent cvolume of soil	$(m_4 - m_1) - (m_3 - m_2)$	g	1.996	1.987	3.284	0.953
Particle density	$m_2 - m_1 / (m_4 - m_1) - (m_3 - m_2)$	Mg/m^3	2.621	2.716	1.813	5.725
Average value		Mg/m^3	2.668		3.769	
Particle density		Mg/m^3	2.668			

Determination of Particle Density

Location	UTM			Job ref.	Vot 71382
				Boer hole/ Pit no.	
Soil description	UTM			Sample No	2
				Depth	
Test method	BS 1377: Part 2: 1990, Clause 8.3			Date	8/8/00
Method pf preparation	Small pyknometer method				
Small/large pyknometer					

Specimen reference			1	2	3	4
Pyknmeter number			1674	1822	1436	1413
Mass of bottle + soil + water	m_3	g	83.520	83.278	79.858	79.697
Mass of bottle + soil	m_2	g	34.438	35.877	32.378	29.726
Mass of bottle full of water	m_4	g	80.314	79.661	76.498	76.567
Mass of bottle	m_1	g	29.188	29.943	26.926	24.644
Mass of soil	$m_2 - m_1$	g	5.250	5.934	5.452	5.082
Mass of water in full bottle	$m_4 - m_1$	g	51.126	49.718	49.572	51.923
Mass of water used	$m_3 - m_2$	g	49.082	47.401	47.480	49.971
Mass of water equivalent cvolume of soil	$(m_4 - m_1) - (m_3 - m_2)$	g	2.044	2.317	2.092	1.952
Particle density	$m_2 - m_1 / (m_4 - m_1) - (m_3 - m_2)$	Mg/m^3	2.568	2.561	2.606	2.603
Average value		Mg/m^3	2.565		2.605	
Particle density		Mg/m^3	2.605			

Determination of Particle Density

Location	Kulai	Job ref. Vot 71382
		Boer hole/ Pit no.
Soil description	Kulai	Sample No 3
		Depth
Test method	BS 1377: Part 2: 1990, Clause 8.3	Date 8/8/00
Method pf preparation	Small pyknometer method	
Small/large pyknometer		

Specimen reference			1	2	3	4
Pyknmeter number			1385	1465	1477	1601
Mass of bottle + soil + water	m_3	g	79.244	82.169	81.219	84.727
Mass of bottle + soil	m_2	g	31.905	32.747	31.946	37.531
Mass of bottle full of water	m_4	g	76.021	78.083	77.429	80.694
Mass of bottle	m_1	g	26.706	26.167	25.940	31.149
Mass of soil	$m_2 - m_1$	g	5.199	6.580	6.006	6.382
Mass of water in full bottle	$m_4 - m_1$	g	49.315	51.916	51.489	49.545
Mass of water used	$m_3 - m_2$	g	47.339	49.422	49.273	47.196
Mass of water equivalent cvolume of soil	$(m_4 - m_1) - (m_3 - m_2)$	g	1.976	2.494	2.216	2.349
Particle density	$m_2 - m_1/ (m_4 - m_1) - (m_3 - m_2)$	Mg/m^3	2.631	2.638	2.710	2.717
Average value		Mg/m^3	2.635		2.714	
Particle density		Mg/m^3	2.674			

APPENDIX I

Result of wet and dry sieving and hydrometer sedementation test

Particle Size Distribution (Sieving)

Location	Tg. Pelepas			Job. Ref Borehole/ Pit no.	Vot 71382
Soil description	PLP			Sample No Depth	1
Test Method	BS 1377: Part 2: 1990, Clause 9.2			Date	7/9/200
Initial dry mass	m_1	50	g		
BS test sieve		Mass retained (g) actual	corrected (m)	Percentage retained $(m/m_1) \times 100$	Cumulative percentage passing
75	mm	0	0	0	100
63	mm	0	0	0	100
50	mm	0	0	0	100
37.5	mm	0	0	0	100
28	mm	0	0	0	100
20	mm	0	0	0	100
Passing 20 mm	m_2	50			
total	(checked with m_1)				
riffled	m_3	50			
riffled and wash	m_4	4.012			
Correction factor	(m_2/m_3)	1			
14	mm	0	0	0	100
10	mm	0	0	0	100
6.3	mm	0	0	0	100
Passing 6.3 mm	m_5	4.012			
total	(checked with m4)				
riffled	m_6	4.012			
Correction factor $(m_2/m_3 \times m_5/m_6)$					
5	mm	0.000	0.000	0.000	100.000
3.35	mm	0.000	0.000	0.000	100.000
2	mm	0.004	0.008	0.016	99.984
1.18	mm	0.454	0.908	1.816	98.168
600	μm	1.024	2.048	4.096	94.072
425	μm	0.514	1.028	2.056	92.016
300	μm	0.354	0.708	1.416	90.600
212	μm	0.304	0.608	1.216	89.384
150	μm	0.204	0.408	0.816	88.568
63	μm	1.124	2.248	4.496	84.072
Passing 63 um	m_F or m_E	0.030			
Total (check with m_6)			(m_1)		

Particle Size Distribution (Sieving)

Location	UTM				Job. Ref Borehole/ Pit no.	Vot 71382
Soil description	UTM				Sample No Depth	2
Test Method	BS 1377: Part 2: 1990, Clause 9.2				Date	7/9/200
Initial dry mass		m_1	50	g		
BS test sieve			Mass retained (g) actual	corrected (m)	Percentage retained $(m/m_1) \times 100$	Cumulative percentage passing
75	mm		0	0	0	100
63	mm		0	0	0	100
50	mm		0	0	0	100
37.5	mm		0	0	0	100
28	mm		0	0	0	100
20	mm		0	0	0	100
Passing 20 mm		m_2	50			
total	(checked with m_1)					
riffled		m_3	50			
riffled and wash		m_4	2.535			
Correction factor	(m_2/m_3)		1			
14	mm		0	0	0	100
10	mm		0	0	0	100
6.3	mm		0	0	0	100
Passing 6.3 mm		m_5	2.535			
total	(checked with m4)					
riffled		m_6	2.535			
Correction factor $(m_2/m_3 \times m_5/m_6)$						
5	mm		0.000	0.000	0.000	100.000
3.35	mm		0.000	0.000	0.000	100.000
2	mm		0.026	0.052	0.104	99.986
1.18	mm		0.056	0.112	0.224	99.672
600	µm		0.146	0.292	0.584	99.088
425	µm		0.106	0.212	0.424	98.664
300	µm		0.106	0.212	0.424	98.240
212	µm		0.166	0.332	0.664	97.576
150	µm		0.246	0.492	0.984	96.592
63	µm		1.326	2.652	5.304	91.288
Passing 63 um		m_F or m_E	0.357			
Total (check with m_6)				(m_1)		

Particle Size Distribution (Sieving)

Location	KULAI			Job. Ref Borehole/ Pit no.		Vot 71382
Soil description	KULAI			Sample No Depth		3
Test Method	BS 1377: Part 2: 1990, Clause 9.2			Date		7/9/200
Initial dry mass	m_1	50	g			
BS test sieve		Mass retained (g)		Percentage retained $(m/m_1) \times 100$	Cumulative percentage passing	
		actual	corrected (m)			
75	mm	0	0	0	100	
63	mm	0	0	0	100	
50	mm	0	0	0	100	
37.5	mm	0	0	0	100	
28	mm	0	0	0	100	
20	mm	0	0	0	100	
Passing 20 mm	m_2	50				
total (checked with m_1)						
riffled	m_3	50				
riffled and wash	m_4	3.857				
Correction factor (m_2/m_3)		1				
14	mm	0	0	0	100	
10	mm	0	0	0	100	
6.3	mm	0	0	0	100	
Passing 6.3 mm	m_5	3.857				
total (checked with m4)						
riffled	m_6	3.857				
Correction factor $(m_2/m_3 \times m_5/m_6)$						
5	mm	0.000	0.000	0.000	100.000	
3.35	mm	0.000	0.000	0.000	100.000	
2	mm	0.024	0.048	0.096	99.904	
1.18	mm	0.094	0.188	0.376	99.528	
600	μm	0.244	0.488	0.976	98.552	
425	μm	0.144	0.288	0.576	97.976	
300	μm	0.244	0.488	0.976	97.000	
212	μm	0.324	0.648	1.296	95.704	
150	μm	0.464	0.928	1.859	93.848	
63	μm	1.874	3.748	7.496	86.352	
Passing 63 um	m_F or m_E	0.445				
Total (check with m_6)			(m_1)			

Particle size distribution (hydrometer sedementation)

Location	Tg. Pelepas	Job ref.	Vot 71382
		Borehole/ Pit no.	
Soil description	PLP	Sample No.	1
		Depth	
Test Method	BS 1377: Part 2: 1990, Clause 9.6	Date	27/8/2000
Method of preparation			

CALIBRATION AND SAMPLE DATA

Hydrometer No.		22108	
Meniscus correction	C_m	0.50	
Reading in dispersant	R_o'	1.0005	
Calbration equation	$H_r = 222.93 - 3.997R_h$		
Dry mass soil	m	50	g
Particle density measured/assumed*	ρ_s	2.67	Mg/m^3
Viscosity of water at 28°C	η	0.835	mPa.s

PRETREATMENT

Pretreated with			
Initial dry mass of sample	m_o		g
Dry mass after pretreatment	m		g
Pretreatment loss	m_o - m		g
			%

TEST DATA

Date	Time (start at 12.00 p.m)	Elapsed time t (min)	Temperature T°C	Reading R_h'	$R_h + C_m$ R_h	Effective depth H_r (mm)	Particle diameter D (mm)	$R_h' - R_o'$ R_d	Percentage finer than D K (%)
26/8/2000	12.00	0.5	28.0	1.0250	25.5	121.01	0.0608	24.5	78.34
	12.01	1.0	28.0	1.0245	25.0	123.01	0.0434	24.0	76.74
	12.02	2.0	28.0	1.0241	24.6	124.60	0.0309	23.6	75.46
	12.04	4.0	28.0	1.0225	23.0	131.00	0.0224	22.0	70.35
	12.08	8.0	28.0	1.0212	21.7	136.20	0.0161	20.7	66.19
	12.30	30.0	28.0	1.0193	19.8	143.79	0.0086	18.8	60.11
	14.00	120.0	28.0	1.0165	17.0	154.98	0.0044	16.0	51.56
	20.00	480.0	28.5	1.0130	13.5	168.97	0.0023	12.5	39.97
27/8/2000	12.00	1440.0	28.5	1.0130	13.5	168.97	0.0013	12.5	39.97

Particle size distribution (hydrometer sedementation)

Location	UTM	Job ref.	Vot 71382
		Borehole/ Pit no.	
Soil description	UTM	Sample No.	2
		Depth	
Test Method	BS 1377: Part 2: 1990, Clause 9.6	Date	29/8/2000
Method of preparation			

CALIBRATION AND SAMPLE DATA

Hydrometer No.		1	
Meniscus correction	C_m	0.50	
Reading in dispersant	R_o'	1.0005	
Calbration equation	$H_r = 222.93 - 3.997R_h$		
Dry mass soil	m	50	g
Particle density measured/assumed*	ρ_s	2.6	Mg/m^3
Viscosity of water at 28°C	η	0.835	mPa.s

PRETREATMENT

Pretreated with		
Initial dry mass of sample	m_o	g
Dry mass after pretreatment	m	g
Pretreatment loss	m_o - m	g
		%

TEST DATA

Date	Time (start at 9.08 a.m)	Elapsed time t (min)	Temperature T°C	Reading R_h'	$R_h + C_m$ R_h	Effective depth H_r (mm)	Particle diameter D (mm)	$R_h' - R_o'$ R_d	Percentage finer than D K (%)
29/8/2000	9.08	0.5	28.0	1.0265	27.0	115.01	0.0606	26.0	84.50
	9.09	1.0	28.0	1.0263	26.8	115.81	0.0430	25.8	83.85
	9.10	2.0	28.0	1.0260	26.5	117.01	0.0306	25.5	82.88
	9.12	4.0	28.0	1.0250	25.5	121.01	0.0220	24.5	79.63
	9.16	8.0	28.0	1.0236	24.1	126.60	0.0159	23.1	75.08
	9.38	30.0	28.0	1.0204	20.9	139.39	0.0086	19.9	64.68
	11.08	120.0	28.0	1.0165	17.0	154.98	0.0045	16.0	52.00
	5.08	480.0	28.0	1.0115	12.0	174.97	0.0024	11.0	35.75
30/8/2000	9.08	1440.0	28.0	1.0081	8.6	188.56	0.0014	7.6	24.70

Particle size distribution (hydrometer sedementation)

Location	KULAI	Job ref.	Vot 71382
		Borehole/ Pit no.	
Soil description	KULAI	Sample No.	3
		Depth	
Test Method	BS 1377: Part 2: 1990, Clause 9.6	Date	1/9/00
Method of preparation			

CALIBRATION AND SAMPLE DATA

Hydrometer No.		22108	
Meniscus correction	C_m	0.50	
Reading in dispersant	R_o'	1.0003	
Calbration equation	$H_r = 222.93 - 3.997R_h$		
Dry mass soil	m	50	g
Particle density measured/assumed*	ρ_S	2.67	Mg/m^3
Viscosity of water at 28°C	η	0.835	mPa.s

PRETREATMENT

Pretreated with			
Initial dry mass of sample	m_o		g
Dry mass after pretreatment	m		g
Pretreatment loss	m_o - m		g
			%

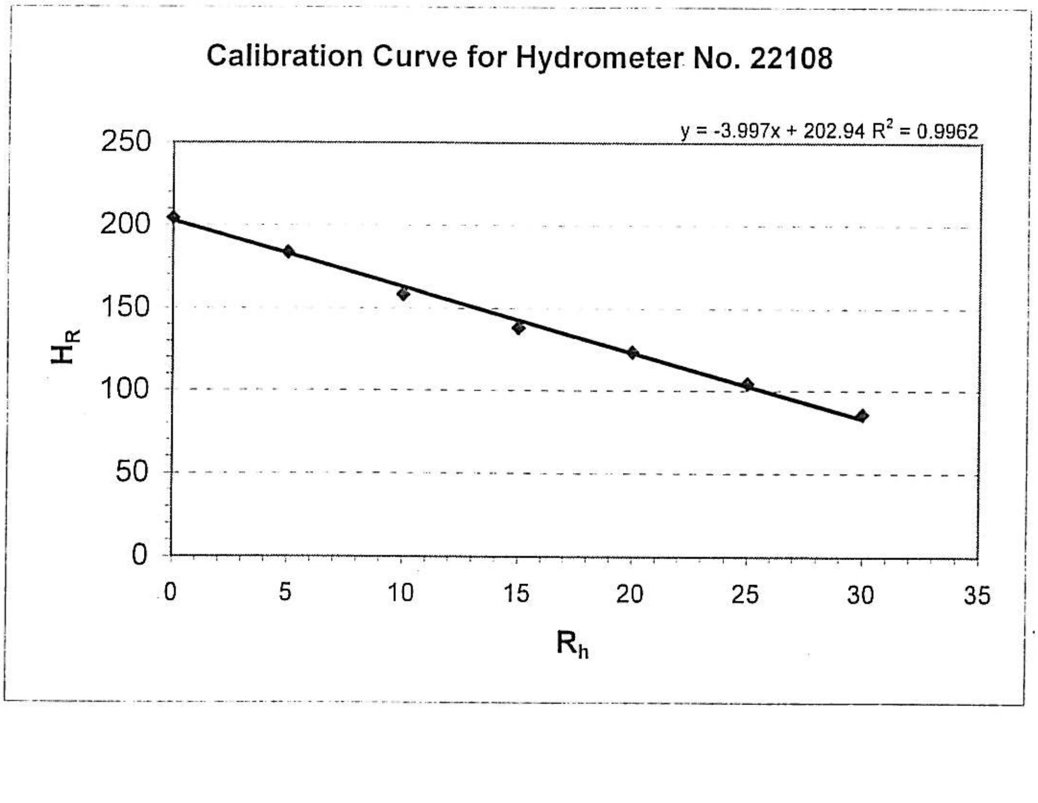
TEST DATA

Date	Time (start at 9.25 a.m)	Elapsed time t (min)	Temperature T°C	Reading R_h'	$R_h + C_m$ R_h	Effective depth H_r (mm)	Particle diameter D (mm)	$R_h' - R_o'$ R_d	Percentage finer than D K (%)
1/9/00	9.25	0.5	28.5	1.0266	26.9	115.41	0.0591	26.3	84.10
	9.26	1.0	28.5	1.0258	26.1	118.61	0.0424	25.5	81.54
	9.27	2.0	28.5	1.0255	25.8	119.81	0.0301	25.2	80.58
	9.29	4.0	28.5	1.0255	25.8	119.81	0.0213	25.2	80.58
	9.33	8.0	28.5	1.0250	25.3	121.80	0.0152	24.7	78.98
	9.55	30.0	28.5	1.0230	23.3	129.80	0.0081	22.7	72.59
	11.25	120.0	28.0	1.0281	18.4	149.39	0.0044	17.8	56.92
	5.25	480.0	27.5	1.0110	11.3	177.76	0.0024	10.7	34.21
2/9/00	9.25	1440.0	28.0	1.0005	0.8	219.73	0.0015	0.2	0.64

Hydrometer No. 22108

Mass	=	72.033	g
N	=	20	mm
h	=	156	mm
V_h	=	72.033	ml
L	=	310	
H	=	$N + d_1, N + d_2, \ldots, N + d_7$	

R_h	Distance from lowest mark, d (mm)	H	H_R
-5	140.00	160.00	225.59
0	118.76	138.76	204.35
5	98.00	118.00	183.59
10	72.66	92.66	158.25
15	52.70	72.70	138.29
20	38.00	58.00	123.59
25	19.00	39.00	104.59
30	0.00	20.00	85.59



APPENDIX J

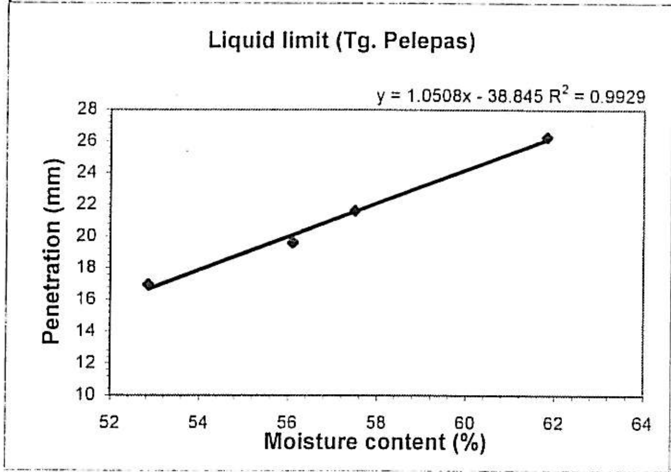
Result of Atterberg Limit test for unstabilized soil

Liquid Limit (cone penetrometer) and plastic limit

Location	Tg. Pelepas	Job ref.	Vot 71382
		Borehole/ Pit no.	
Soil description	PLP	Sample No.	1
		Depth	
Test method	BS 1377: Part 2: 1990, Clause 4.4	Date	6.6.2000

PLASTIC LIMIT	Test No.	1	2	3	4
Container No.		30	88	140	103
Mass of wet soil + container	g	63.60	64.40	64.90	70.79
Mass of dry soil + container	g	63.21	64.03	64.49	70.39
Mass of container	g	61.42	62.41	62.85	68.79
Mass of moisture	g	0.39	0.37	0.41	0.40
Mass of dry soil	g	1.79	1.62	1.64	1.60
Moisture content	%	21.79	22.84	25.00	25.00
Average moisture content	%	23.66			

LIQUID LIMIT	Test No.	1			2			3			4		
Initial dial gauge reading	mm	0	0	0	0	0	0	0	0	0	0	0	0
Final dial gauge reading	mm	17	17	17	20	18	21	22	22	20	26.5	24.4	27.9
Average penetration	mm	16.97			19.63			21.60			26.27		
Container no.		5			149			82			21		
Mass of wet soil + container	g	80.52			79.69			95.57			90.15		
Mass of dry soil + container	g	74.11			72.98			87.86			79.14		
Mass of container	g	61.98			61.02			74.45			61.33		
Mass of moisture	g	6.41			6.71			7.71			11.01		
Mass of dry soil	g	12.13			11.96			13.41			17.81		
Moisture content	%	52.84			56.10			57.49			61.82		



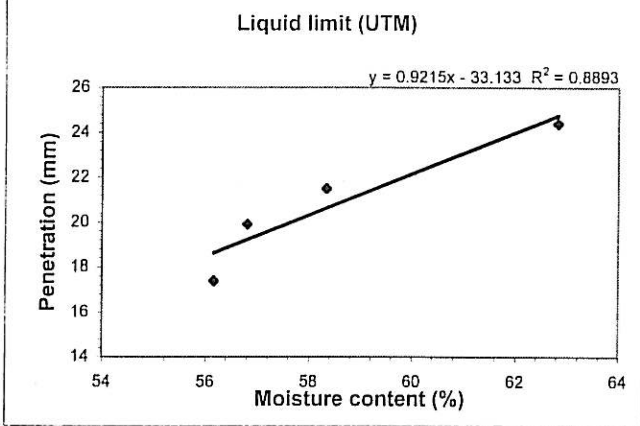
Sample preparation		
as received washed on 425 μm sieve		%
air dried at		°C
oven dried at		°C
Proportion retained on 425 μm sieve		%
Liquid limit	56.00	%
Plastic limit	23.66	%
Plasticity Index	**32.34**	

Liquid Limit (cone penetrometer) and plastic limit

Location	UTM			Job ref.	Vot 71382
Soil description	UTM			Borehole/ Pit no.	
				Sample No.	2
				Depth	
Test method		BS 1377: Part 2: 1990, Clause 4.4		Date	6.6.2000

PLASTIC LIMIT	Test No.	1	2	3	4
Container No.		28A	68A		
Mass of wet soil + container	g	12.45	11.49		
Mass of dry soil + container	g	11.84	11.03		
Mass of container	g	9.49	9.51		
Mass of moisture	g	0.61	0.46		
Mass of dry soil	g	2.35	1.52		
Moisture content	%	25.96	30.26		
Average moisture content	%		28.11		

LIQUID LIMIT	Test No.	1	2	3	4
Initial dial gauge reading	mm	0	0	0	0
Final dial gauge reading	mm	17.4	19.9	21.5	24.4
Average penetration	mm	17.40	19.90	21.50	24.40
Container no.		39	115A	90A	77A
Mass of wet soil + container	g	20.87	19.79	22	20.92
Mass of dry soil + container	g	16.95	16.2	17.38	16.51
Mass of container	g	9.97	9.88	9.46	9.49
Mass of moisture	g	3.92	3.59	4.62	4.41
Mass of dry soil	g	6.98	6.32	7.92	7.02
Moisture content	%	56.16	56.80	58.33	62.82



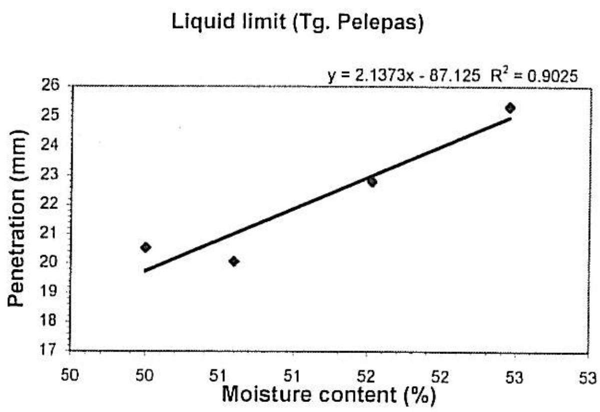
Sample preparation		
as received washed on 425 μm sieve		%
air dried at		°C
oven dried at		°C
Proportion retained on 425 μm sieve		%
Liquid limit	57.65	%
Plastic limit	28.11	%
Plasticity Index	**29.54**	

Liquid Limit (cone penetrometer) and plastic limit

Location	KULAI	Job ref.	Vot 71382
Soil description	KUL	Borehole/ Pit no.	
		Sample No.	3
		Depth	
Test method	BS 1377: Part 2: 1990, Clause 4.4	Date	6.6.2000

PLASTIC LIMIT	Test No.	1	2	3	4
Container No.		107	115	65	112
Mass of wet soil + container	g	61.83	58.44	69.50	62.00
Mass of dry soil + container	g	61.28	57.95	69.14	61.35
Mass of container	g	59.28	56.25	67.89	59.11
Mass of moisture	g	0.55	0.49	0.36	0.65
Mass of dry soil	g	2.00	1.70	1.25	2.24
Moisture content	%	27.50	28.82	28.80	29.02
Average moisture content	%			28.54	

LIQUID LIMIT	Test No.	1			2			3			4		
Initial dial gauge reading	mm	0	0	0	0	0	0	0	0	0	0	0	0
Final dial gauge reading	mm	18.9	18.8	22.5	19	23.3	26.1	20.5	19.8	21.3	24	26.5	25.5
Average penetration	mm	20.07			22.80			20.53			25.33		
Container no.		150			83			118			39		
Mass of wet soil + container	g	75.86			90.97			75.08			89.42		
Mass of dry soil + container	g	70.79			81.69			70.29			79.68		
Mass of container	g	60.77			63.68			60.71			61.11		
Mass of moisture	g	5.07			9.28			4.79			9.74		
Mass of dry soil	g	10.02			18.01			9.58			18.57		
Moisture content	%	50.60			51.53			50.00			52.45		



Sample preparation		
as received washed on 425 μm sieve		%
air dried at		°C
oven dried at		°C
Proportion retained on 425 μm sieve		%
Liquid limit	50.12	%
Plastic limit	28.54	%
Plasticity Index	**21.58**	

APPENDIX K

Result of organic content test

Determination of Loss On Ignition (LOI)
BS 1377: Part 3, Clause 3

Sample	Tg. Pelepas		UTM		KULAI	
Crucible No.	P1	P2	U1	U2	K1	K2
Mass of crucible, m_c	20.761	20.603	18.738	21.342	20.597	11.627
Mass of crucible and dry soil, m_3	25.601	25.479	23.781	26.578	25.804	16.727
Mass of crucible and soil after ignition, m_4	25.319	25.197	23.707	26.499	25.779	16.704
$LOI = \left[\frac{m_3 - m_4}{m_3 - m_c}\right] x100$	5.826	5.783	1.467	1.509	0.480	0.451

APPENDIX L

Result of suitability of lime

Determination of the suitability of the lime

Material		Job ref.	Vot 71382
Hydrated Lime	$Ca(OH)_2$		
Test method	BS 1924: Part 2: 1990, Clause 5.4.6	Date	2.8.2000
Test No		1	2
pH value of the suspension		12.35	12.40
Temperature		26.70	24.80
pH corrected to 25°C	($pH_{25} = pH_T + 0.03(T - 25)$)	12.40	12.39
pH_{25} in the range of 12.35 - 12.40	* Lime is suitable for use		

APPENDIX M

Result of available lime content test

Determination of available lime

Material Hydrated Lime $Ca(OH)2$		Job ref.	Vot 71382
Test method	BS 6463: Part 2: 1984 (Method 20)	Date	2.8.2000
Test No.		1	2
Mass of sample, m	g	1	1
Volume of the hydrichloric acid, V_o	mL	23.8	2.0
Volume of the hydrichloric acid, V_1	mL	48.6	25.2
Volume of the titration, $V = V_1 - V_o$	mL	24.8	23.2
Percentage available lime as CaO	$\frac{2.804\,V}{m}$	69.54	65.05
Percentage available lime as $Ca(OH)_2$	$\frac{3.705\,V}{m}$	91.88	85.96
Average CaO		67.296	
Average $Ca(OH)_2$		88.92	

APPENDIX N

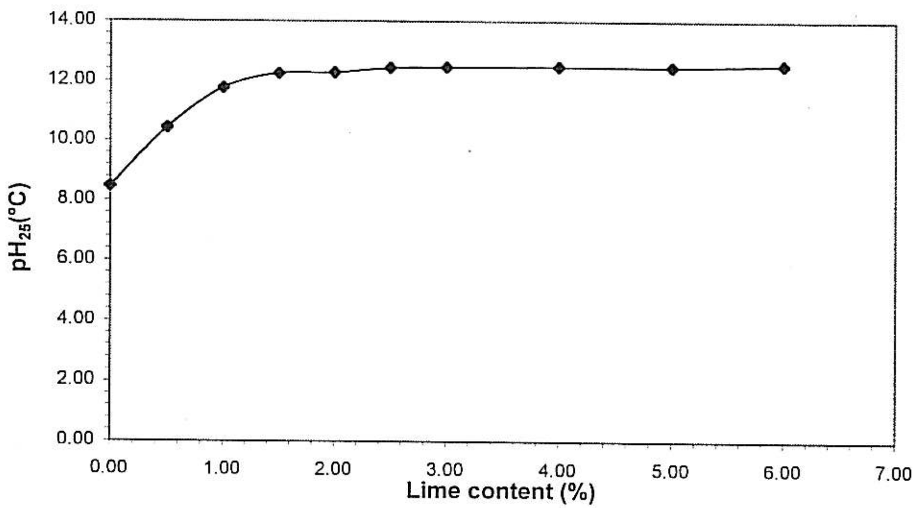
Result of Initial Consumption of Lime (ICL) test

Laboratory	Makmal Kimia JGP	Material	PLP
Operator	Mustafa	Site	Tg. Pelepas
Job	Vot 71382	Date	17.8.200

pH of saturated solution	pH_T	
Temperature (°C)	T	
pH corrected to 25°C	$(pH_{25} = pH_T + 0.03(T - 25))$	

Lime content	%	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	6.00
pH value of suspension		8.42	10.39	11.74	12.21	12.25	12.40	12.43	12.45	12.44	12.49
Temperature	°C	26.50	26.10	26.00	26.10	25.80	26.30	25.80	26.00	26.10	26.30
pH corrected to 25°C		8.47	10.42	11.77	12.24	12.27	12.44	12.45	12.48	12.47	12.53

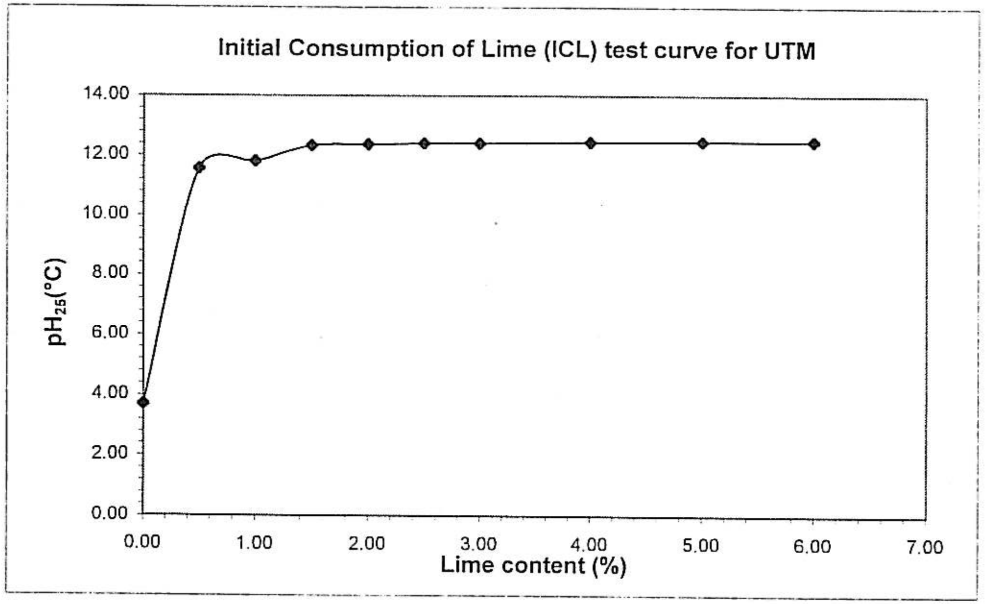
Initial Consumption of Lime (ICL) test curve for Tg. Pelepas



Laboratory	Makmal Kimia JGP	Material	UTM
Operator	Mustafa	Site	UTM
Job	Vot 71382	Date	17.8.200

pH of saturated solution	pH_T	
Temperature (°C)	T	
pH corrected to 25°C	$(pH_{25} = pH_T + 0.03(T - 25)$	

Lime content	%	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	6.00
pH value of suspension		3.64	11.52	11.77	12.26	12.30	12.34	12.35	12.40	12.40	12.42
Temperature	°C	27.00	26.10	26.10	27.00	27.00	27.20	26.70	26.70	27.10	27.10
pH corrected to 25°C		3.70	11.55	11.80	12.32	12.36	12.41	12.40	12.45	12.46	12.48

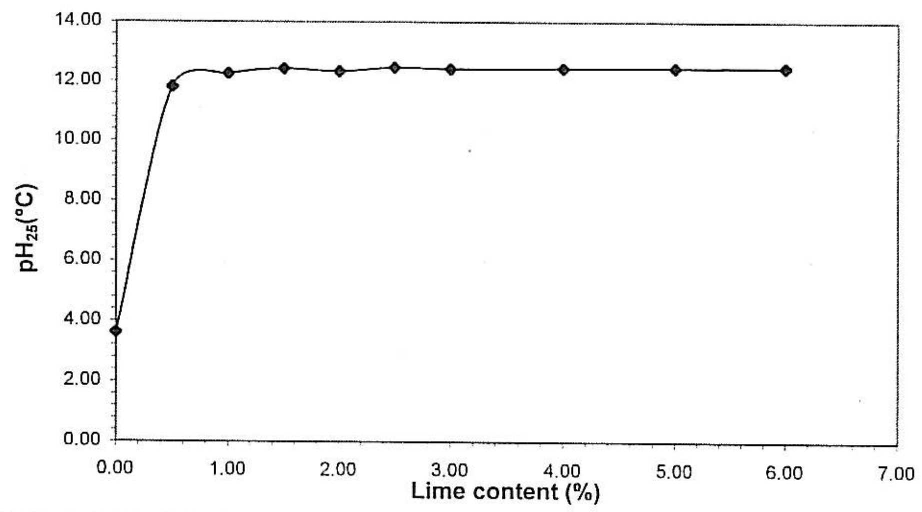


Laboratory	Makmal Kimia JGP	Material	Kulai
Operator	Mustafa	Site	KUL
Job	Vot 71382	Date	17.8.200

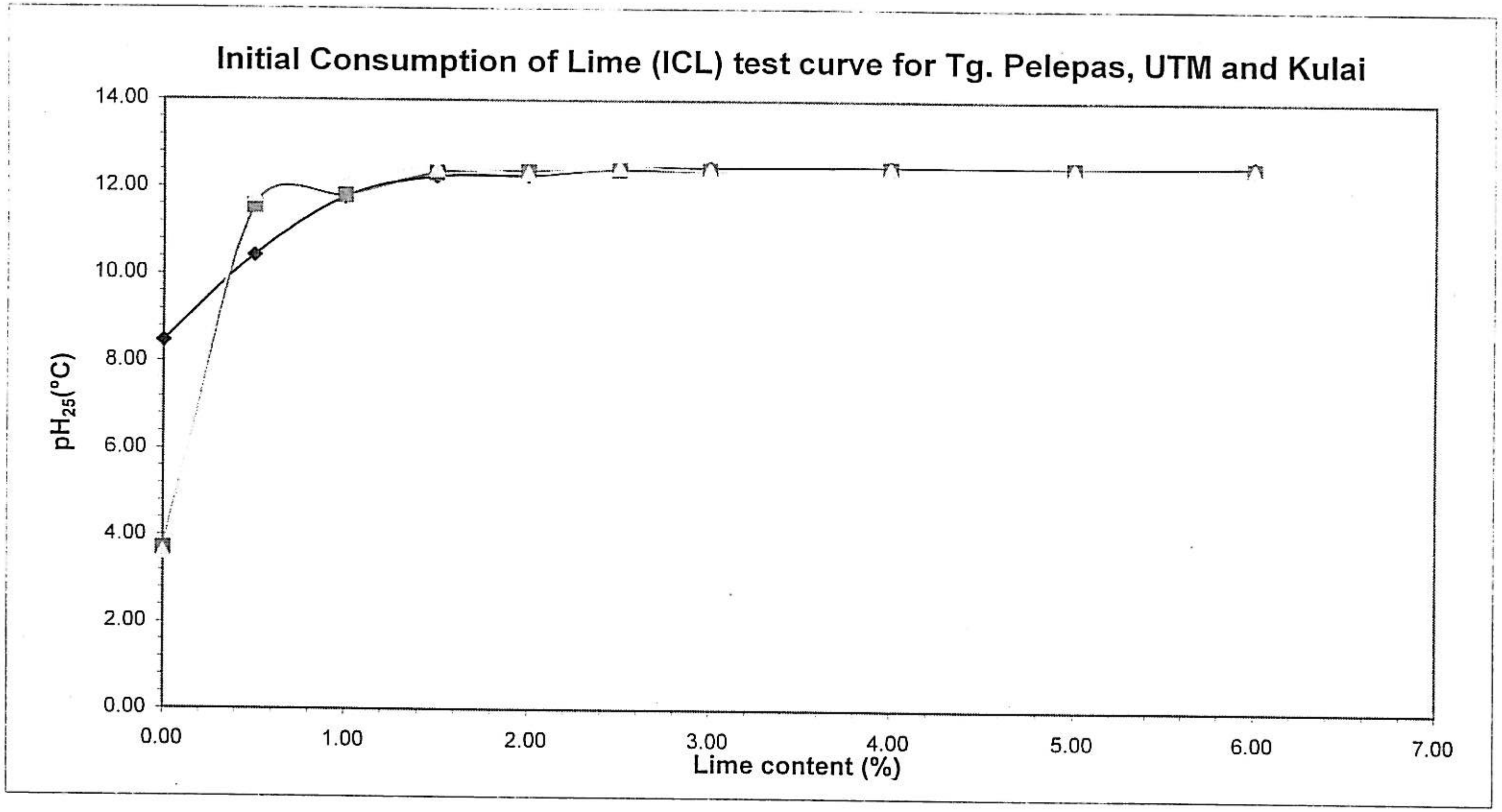
pH of saturated solution	pH_T	
Temperature (°C)	T	
pH corrected to 25°C	($pH_{25} = pH_T + 0.03(T - 25)$)	

Lime content	%	0.00	0.50	1.00	1.50	2.00	2.50	3.00	4.00	5.00	6.00
pH value of suspension		3.59	11.77	12.22	12.37	12.31	12.41	12.39	12.42	12.42	12.44
Temperature	°C	25.60	26.30	26.20	26.50	25.50	26.50	25.70	25.70	25.90	25.90
pH corrected to 25°C		3.61	11.81	12.26	12.42	12.33	12.46	12.41	12.44	12.45	12.47

Initial Consumption of Lime (ICL) test curve for Kulai



Summary of the Initial Consumption of Lime (ICL) test curve



APPENDIX O

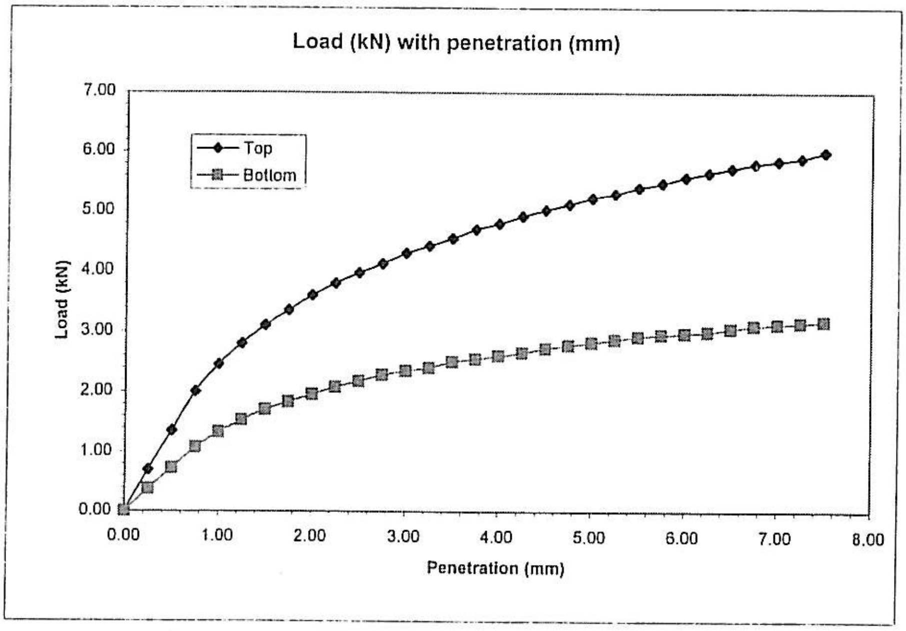
California Bearing Ratio (CBR) test – Unstabilized sample

Location	Tg. Pelepas	Job ref.	Vot 71382
		Borehole/ Pit no.	
Soil description	PLP (m - 8%)	Sample No.	1
		Depth	m
Test method	BS 1377 Part 4: 1990, Clause 7.2.4.4		
		Unsoaked/ soaked	
Force measuring no.		Mean calibration	0.025 kN/div

Penetration of plunger (mm)	Force gauge reading (div)		Force on plunger (kN)		Penetration of plunger (mm)	Force gauge reading (div)		Force on plunger (kN)	
	Top	Bottom	Top	Bottom		Top	Bottom	Top	Bottom
0.00	0.00	0.00	0.00	0.00					
0.25	28.00	15.00	0.70	0.38	4.00	192.00	104.00	4.80	2.60
0.50	54.00	29.00	1.35	0.73	4.25	197.00	106.00	4.93	2.65
0.75	80.00	43.00	2.00	1.08	4.50	201.00	109.00	5.03	2.73
1.00	98.00	53.00	2.45	1.33	4.75	205.00	111.00	5.13	2.78
1.25	112.00	61.00	2.80	1.53	5.00	209.00	113.00	5.23	2.83
1.50	124.00	68.00	3.10	1.70	5.25	212.00	115.00	5.30	2.88
1.75	134.00	73.00	3.35	1.83	5.50	216.00	117.00	5.40	2.93
2.00	144.00	78.00	3.60	1.95	5.75	219.00	118.00	5.48	2.95
2.25	152.00	83.00	3.80	2.08	6.00	223.00	119.00	5.58	2.98
2.50	159.00	87.00	3.98	2.18	6.25	226.00	120.00	5.65	3.00
2.75	165.00	91.00	4.13	2.28	6.50	229.00	122.00	5.73	3.05
3.00	172.00	94.00	4.30	2.35	6.75	232.00	124.00	5.80	3.10
3.25	177.00	96.00	4.43	2.40	7.00	234.00	125.00	5.85	3.13
3.50	182.00	100.00	4.55	2.50	7.25	236.00	126.00	5.90	3.15
3.75	188.00	102.00	4.70	2.55	7.50	240.00	127.00	6.00	3.18

Moisture content after test

Container No.			16	115	CBR (%) value at penetration of		
Mass of wet soil + container, m_2		g	74.49	64.58			
Mass of dry soil + container, m_3		g	73.35	63.66	Side	2.50 mm	5.00 mm
Mass of container, m_1		g	64.50	56.27	Top		
Mass of moisture, $m_2 - m_3$	a	g	1.14	0.92	Bottom		
Mass of dry soil, $m_3 - m_1$	b	g	8.85	7.39	Accepted		
Moisture content	(a/b)*100	%	12.88	12.45	CBR		%
Average moisture content		%	12.67				



APPENDIX P

Unconfined Compressive Strength (UCS) test

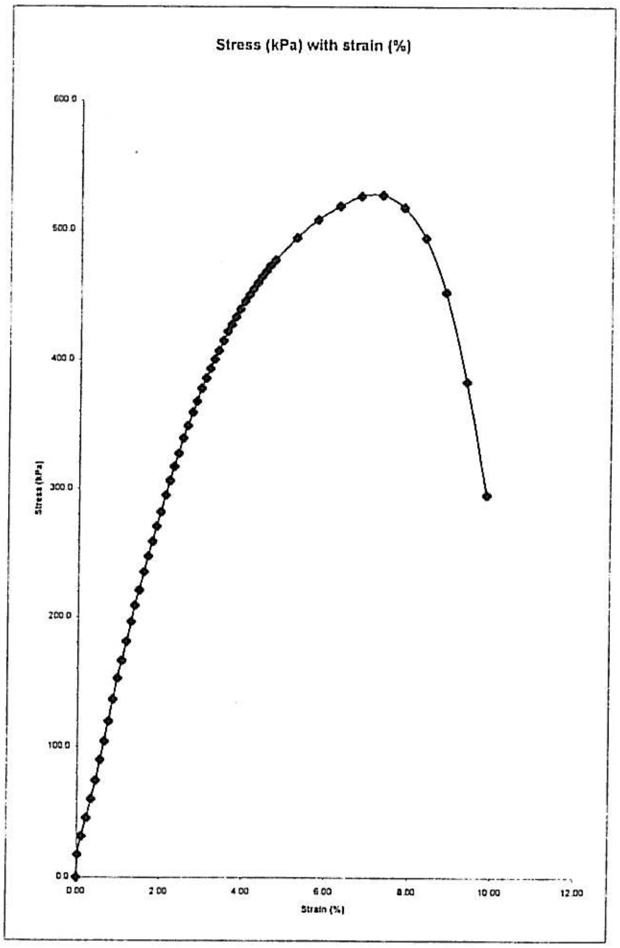
Unconfined Compressive Strength

Job Vot 71382
Sample PLP 1
Specimen No. 1

No	Axial strain %	Axial force N	Dev. Stress kPa	Sigma 1 Eff. kPa
0	0.00	0	0.0	0.0
1	0.02	20	17.6	17.6
2	0.12	36	31.9	31.9
3	0.23	52	46.1	46.1
4	0.34	68	60.2	60.2
5	0.45	85	74.3	74.3
6	0.56	103	90.1	90.1
7	0.66	119	104.4	104.4
8	0.76	137	119.7	119.7
9	0.86	156	136.3	136.3
10	0.97	175	152.7	152.7
11	1.07	190	166.2	166.2
12	1.18	208	180.9	180.9
13	1.29	225	196.1	196.1
14	1.38	240	208.8	208.8
15	1.48	254	220.9	220.9
16	1.59	271	235.2	235.2
17	1.69	286	247.5	247.5
18	1.79	299	259.0	259.0
19	1.89	313	270.7	270.7
20	1.99	326	282.1	282.1
21	2.11	342	295.1	295.1
22	2.21	355	306.2	306.2
23	2.31	368	317.1	317.1
24	2.41	380	327.3	327.3
25	2.52	394	339.1	339.1
26	2.63	406	348.7	348.7
27	2.74	419	359.3	359.3
28	2.84	429	367.9	367.9
29	2.95	442	377.9	377.9
30	3.06	451	385.7	385.7
31	3.16	461	393.4	393.4
32	3.26	469	400.5	400.5
33	3.36	478	407.2	407.2
34	3.47	488	414.9	414.9
35	3.57	496	421.9	421.9
36	3.67	503	427.3	427.3
37	3.77	511	433.2	433.2
38	3.87	518	438.8	438.8
39	3.99	526	444.9	444.9
40	4.09	532	449.9	449.9
41	4.19	538	454.6	454.6
42	4.29	544	459.2	459.2
43	4.39	550	464.0	464.0
44	4.49	556	468.2	468.2
45	4.59	561	472.2	472.2
46	4.71	567	476.7	476.7
47	5.22	591	494.0	494.0
48	5.73	611	507.8	507.8
49	6.26	627	518.4	518.4
50	6.78	640	526.1	526.1
51	7.29	645	527.0	527.0
52	7.81	637	517.4	517.4
53	8.34	611	493.6	493.6
54	8.84	562	452.0	452.0
55	9.35	479	383.1	383.1
56	9.86	372	295.6	295.6

Moisture content

Mass of wet soil + container		m_2	
Mass of dry soil + container		m_3	
Mass of container		m_1	
Mass of moisture	(a)	$m_2 - m_1$	
Mass of dry soil	(b)	$m_3 - m_1$	
Moisture content	(a/b)*100	%	



Vane Shear Strength Test

Sample PLP 4
Specimen No. 1

Spring No.	D_0	D_1	D (D_0 - D_1)	Applied torque T (lb.in)	Undrained Shear Strength S_u (kPa)
S1	281	293	12	0.39	10.03
S2	325	338	13	0.35	9.00
S3	339	366	27	0.50	12.86
S4	3	23	20	0.25	6.43
Average					9.58

Calculation

$$S_u = \frac{T \times 4.536 \times 10^{-3} \times 0.0254}{4.4803 \times 10^{-6}}$$

* T is applied torque based on chart (lb.in)

Moisture content

Mass of wet soil + container		m_2	38.12
Mass of dry soil + container		m_3	28.87
Mass of container		m_1	9.79
Mass of moisture	(a)	$m_2 - m_1$	9.25
Mass of dry soil	(b)	$m_3 - m_1$	19.08
Moisture content	(a/b)*100	%	48.48

APPENDIX Q

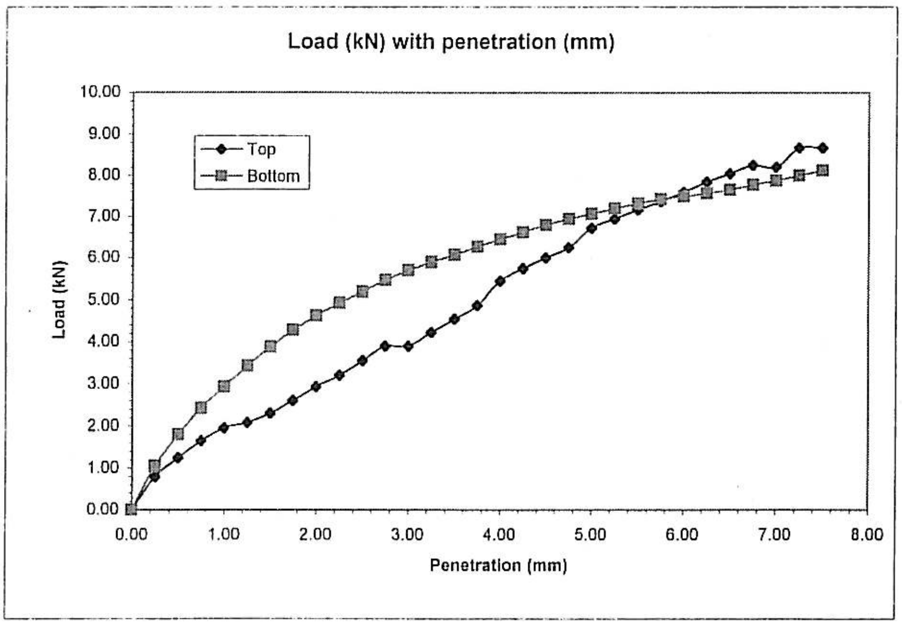
California Bearing Ratio (CBR) test – stabilized sample

Location KULAI		Job ref.	Vot 71382
		Borehole/ Pit no.	
Soil description KUL - 6 (56 days)		Sample No.	6
		Depth	m
Test method	BS 1377 Part 4: 1990, Clause 7.2.4.4		
		Unsoaked/ soaked	
Force measuring no.		Mean calibration	0.025 kN/div

Penetration of plunger (mm)	Force gauge reading (div)		Force on plunger (kN)		Penetration of plunger (mm)	Force gauge reading (div)		Force on plunger (kN)	
	Top	Bottom	Top	Bottom		Top	Bottom	Top	Bottom
0.00	0.00	0.00	0.00	0.00					
0.25	32.00	42.00	0.80	1.05	4.00	218.00	258.00	5.45	6.45
0.50	50.00	72.00	1.25	1.80	4.25	230.00	265.00	5.75	6.63
0.75	66.00	97.00	1.65	2.43	4.50	240.00	272.00	6.00	6.80
1.00	78.00	117.00	1.95	2.93	4.75	250.00	278.00	6.25	6.95
1.25	83.00	137.00	2.08	3.43	5.00	260.00	283.00	6.73	7.08
1.50	92.00	155.00	2.30	3.88	5.25	269.00	288.00	6.95	7.20
1.75	104.00	171.00	2.60	4.28	5.50	278.00	293.00	7.18	7.33
2.00	117.00	185.00	2.93	4.63	5.75	287.00	297.00	7.38	7.43
2.25	128.00	197.00	3.20	4.93	6.00	295.00	300.00	7.60	7.50
2.50	142.00	208.00	3.55	5.20	6.25	304.00	303.00	7.85	7.58
2.75	156.00	219.00	3.90	5.48	6.50	314.00	306.00	8.05	7.65
3.00	169.00	228.00	3.90	5.70	6.75	322.00	311.00	8.25	7.78
3.25	182.00	236.00	4.23	5.90	7.00	330.00	315.00	8.20	7.88
3.50	195.00	243.00	4.55	6.08	7.25	328.00	320.00	8.68	8.00
3.75	203.00	251.00	4.88	6.28	7.50	347.00	325.00	8.68	8.13

Moisture content after test

Container No.			3A	3B	CBR (%) value at penetration of		
Mass of wet soil + container, m_2		g	52.50	48.40			
Mass of dry soil + container, m_3		g	50.10	46.10	Side	2.50 mm	5.00 mm
Mass of container, m_1		g	40.00	36.70	Top		
Mass of moisture, m_2 - m_3	a	g	2.40	2.30	Bottom		
Mass of dry soil, m_3 - m_1	b	g	10.10	9.40	Accepted		
Moisture content	(a/b)*100	%	23.76	24.47	CBR		%
Average moisture content		%	24.12				



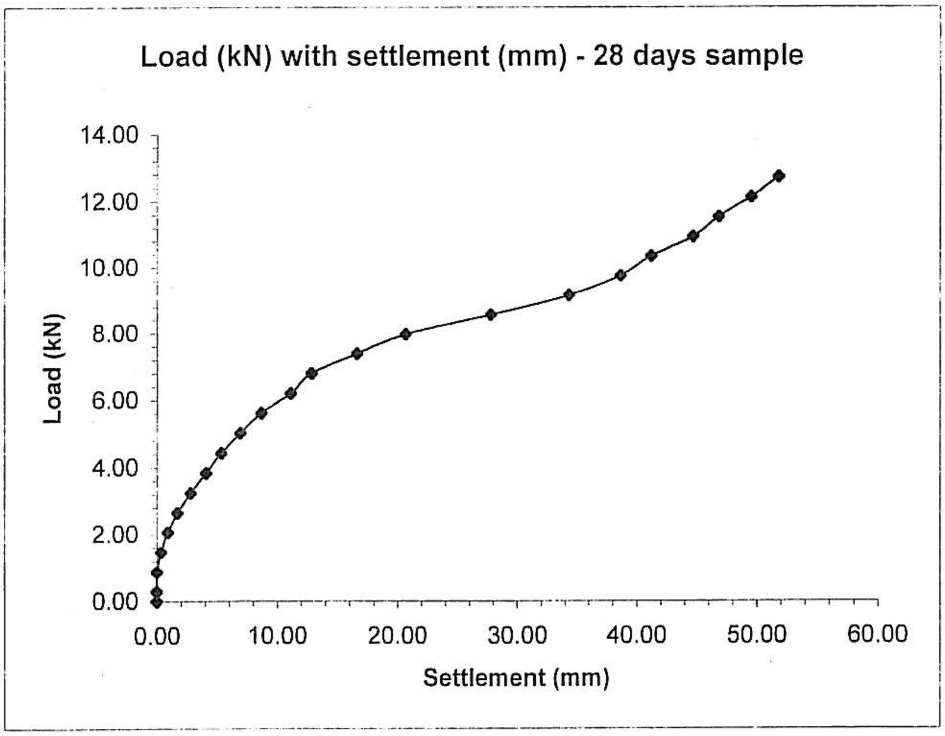
APPENDIX R

Sample of deformation model test result

Model Testing   Deformation model

Sample	Stabilized sample - 28 days		
Self weight	Proving ring	16.76	kg
	Hydraulic jet	5.10	kg
	Pentration cylinder	7.54	kg
Load (self weight)	0.29		kN
Limitation of settlement	2		mm

Load	Settlement (mm)			Average
(kN)	1	2	3	(mm)
0.00	0.00	0.00	0.00	0.00
0.29	0.00	0.00	0.00	0.00
0.88	0.04	0.03	0.01	0.03
1.47	0.62	0.38	0.13	0.38
2.07	1.37	0.94	0.50	0.94
2.66	2.34	1.73	1.12	1.73
3.25	3.60	2.81	2.01	2.81
3.84	5.20	4.11	3.02	4.11
4.43	6.80	5.38	3.95	5.38
5.03	8.64	6.96	5.27	6.96
5.62	10.64	8.70	6.76	8.70
6.21	13.50	11.16	8.82	11.16
6.80	15.65	12.91	10.17	12.91
7.39	20.19	16.65	13.11	16.65
7.99	25.17	20.71	16.24	20.71
8.58	33.20	27.83	22.46	27.83
9.17	39.82	34.38	28.93	34.38
9.76	44.04	38.70	33.35	38.70
10.35	46.61	41.24	35.87	41.24
10.95	50.15	44.76	39.37	44.76
11.54	52.41	46.88	41.34	46.88
12.13	55.36	49.57	43.78	49.57
12.72	57.91	51.84	45.76	51.84



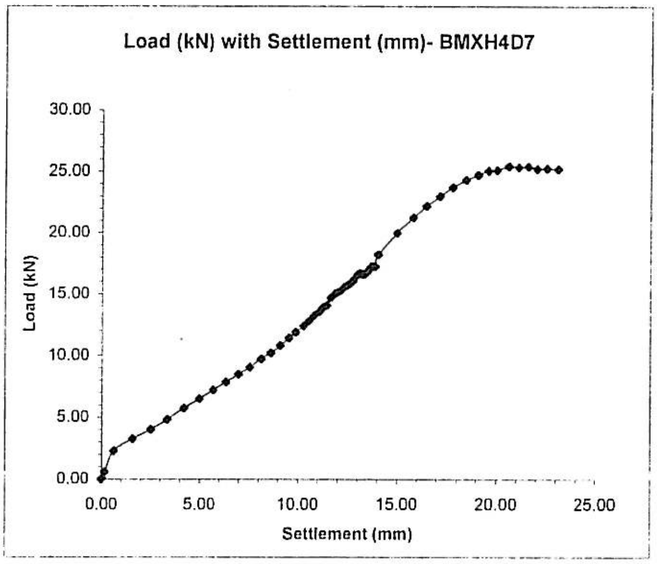
APPENDIX S

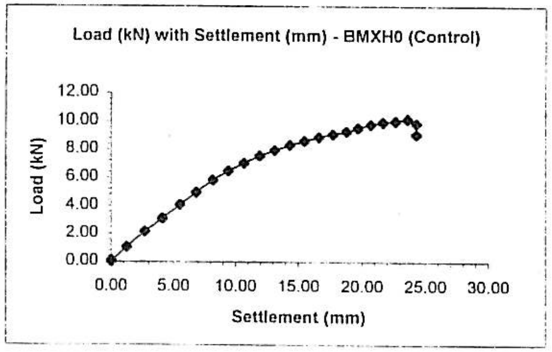
Sample of bearing capacity model test result

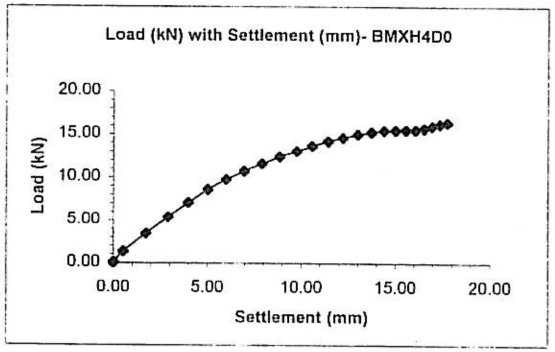
Model Testing

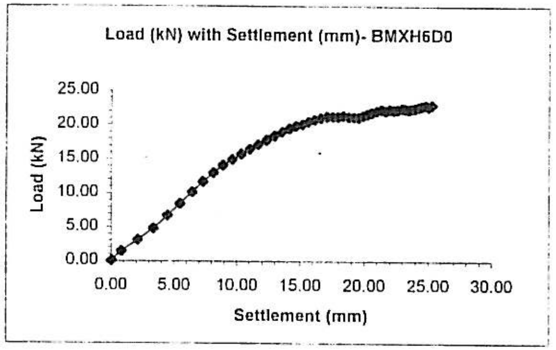
BMXH4D7

Load (kN)	ettlement ((mm)	ettlement ((mm)	Average (mm)
0.00	0.00	0.00	0.00
0.03	0.01	0.01	0.01
0.10	0.05	0.01	0.03
0.60	0.32	0.02	0.17
2.30	1.07	0.21	0.64
3.26	2.06	1.13	1.60
4.03	3.02	2.05	2.54
4.83	3.89	2.85	3.37
5.76	4.76	3.65	4.21
6.50	5.59	4.41	5.00
7.23	6.32	5.10	5.71
7.87	7.01	5.70	6.36
8.50	7.68	6.32	7.00
9.07	8.29	6.84	7.57
9.73	8.92	7.38	8.15
10.23	9.42	7.86	8.64
10.83	9.94	8.28	9.11
11.43	10.43	8.68	9.56
11.87	10.80	8.98	9.89
12.40	11.19	9.39	10.29
12.80	11.52	9.59	10.56
13.23	11.81	9.82	10.82
13.54	12.11	10.04	11.08
13.90	12.33	10.22	11.28
14.10	12.54	10.38	11.46
14.70	12.78	10.55	11.67
15.04	13.03	10.74	11.89
15.17	13.20	10.87	12.04
15.34	13.34	11.01	12.18
15.54	13.49	11.13	12.31
15.67	13.65	11.24	12.45
15.74	13.76	11.32	12.54
15.84	13.85	11.38	12.62
15.94	13.93	11.45	12.69
16.07	14.02	11.51	12.77
16.27	14.11	11.58	12.85
16.57	14.28	11.70	12.99
16.70	14.41	11.79	13.10
16.67	14.50	11.86	13.18
16.67	14.56	11.91	13.24
16.60	14.60	11.96	13.28
16.64	14.63	11.98	13.31
16.64	14.68	12.02	13.35
16.70	14.72	12.03	13.38
16.74	14.74	12.06	13.40
16.77	14.79	12.10	13.45
16.80	14.84	12.12	13.48
16.87	14.88	12.17	13.53
17.00	14.93	12.20	13.57
17.10	14.99	12.24	13.62
17.20	15.05	12.28	13.67
17.20	15.11	12.36	13.74
17.24	15.16	12.38	13.77
17.27	15.21	12.40	13.81
17.27	15.25	12.43	13.84
17.24	15.27	12.46	13.87
17.27	15.31	12.48	13.90
18.24	15.46	12.61	14.04
20.01	16.57	13.41	14.99
21.27	17.52	14.11	15.82
22.21	18.28	14.66	16.47
23.01	19.06	15.23	17.15
23.74	19.81	15.79	17.80
24.34	20.56	16.37	18.47
24.74	21.27	16.88	19.08
25.07	21.88	17.32	19.60
25.14	22.38	17.68	20.03
25.44	23.05	18.16	20.61
25.37	23.63	18.59	21.11
25.41	24.20	19.01	21.61
25.24	24.69	19.40	22.05
25.27	25.24	19.81	22.53
25.21	25.89	20.32	23.11




Load (kN) with Settlement (mm) - BMXH0 (Control)
Load (kN)
Settlement (mm)


Load (kN) with Settlement (mm)- BMXH4D0
Load (kN)
Settlement (mm)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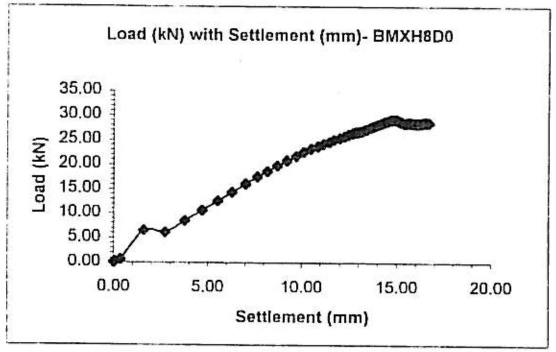
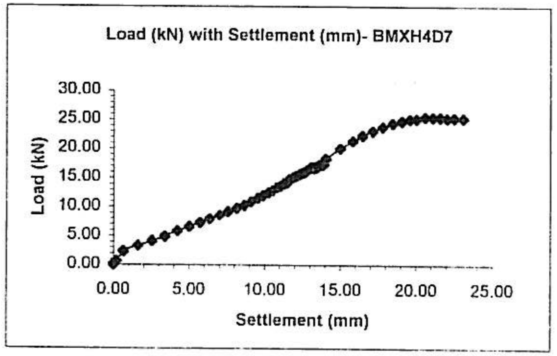
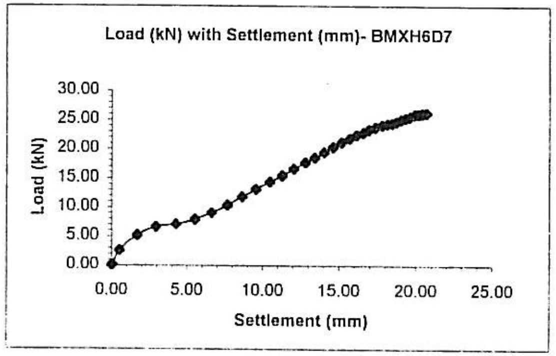
Load (kN)
Settlement (mm)


Load (kN) with Settlement (mm)- BMXH8D0
Load (kN)
Settlement (mm)


Load (kN) with Settlement (mm)- BMXH4D7
Load (kN)
Settlement (mm)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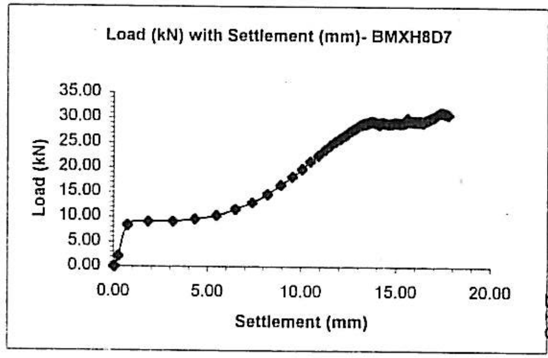
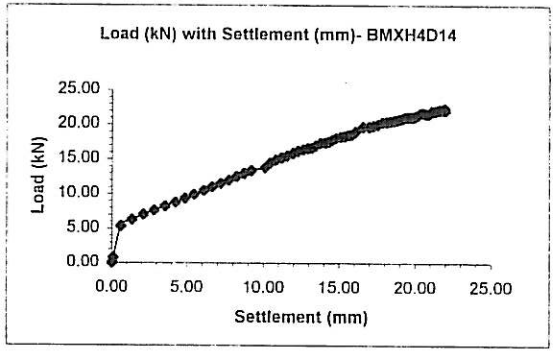
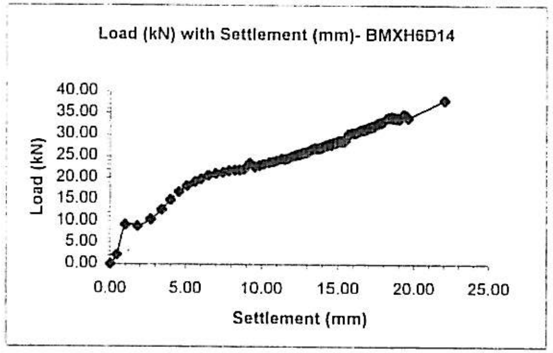
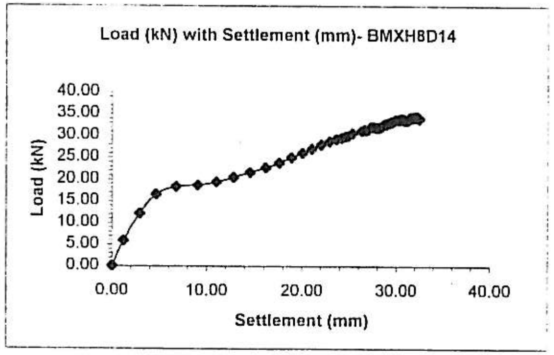
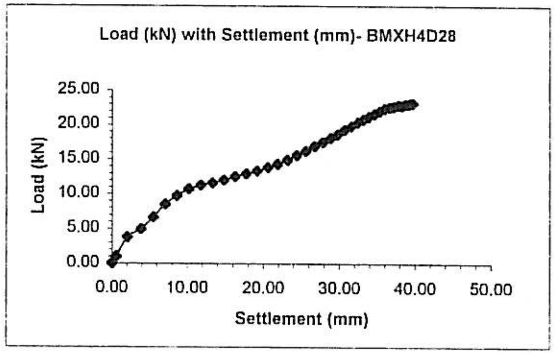
Load (kN)
Settlement (mm)

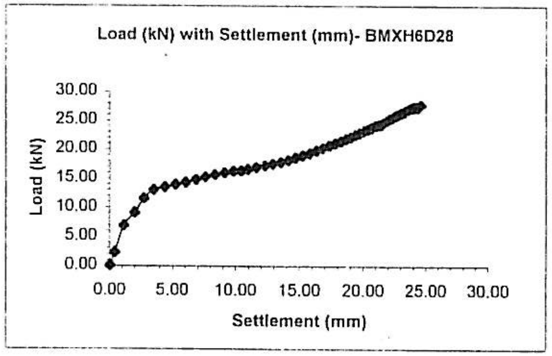

Load (kN) with Settlement (mm)- BMXH8D7
Load (kN)
Settlement (mm)

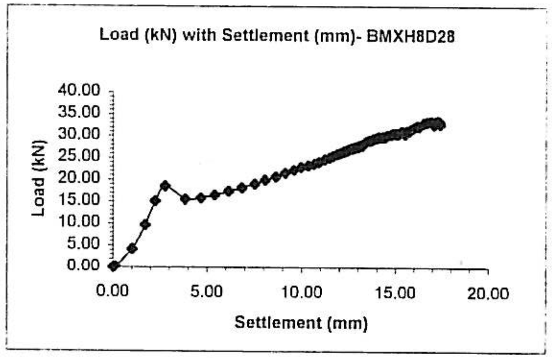

Load (kN) with Settlement (mm)- BMXH4D14
Load (kN)
Settlement (mm)


Load (kN) with Settlement (mm)- BMXH6D14
Load (kN)
Settlement (mm)


Load (kN) with Settlement (mm)- BMXH8D14
Load (kN)
Settlement (mm)


Load (kN) with Settlement (mm)- BMXH4D28
Load (kN)
Settlement (mm)

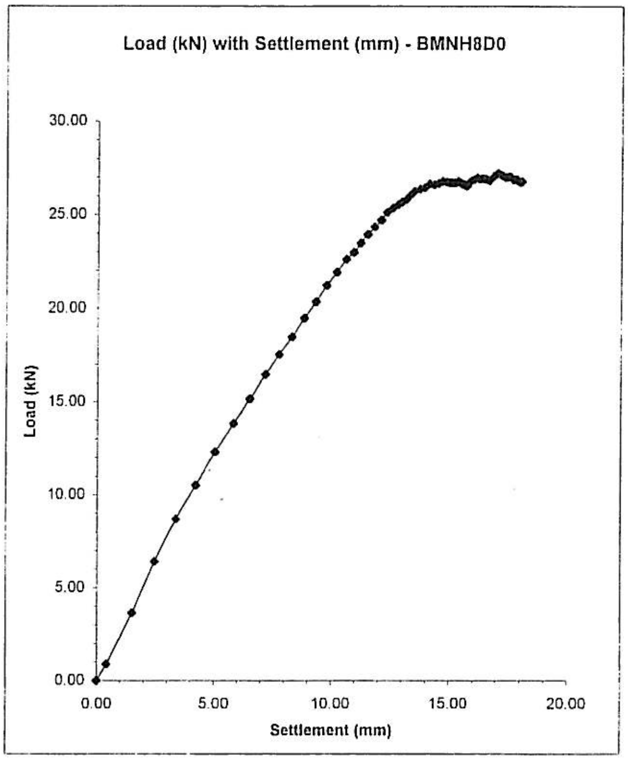

Load (kN) with Settlement (mm)- BMXH6D28
Load (kN)
Settlement (mm)

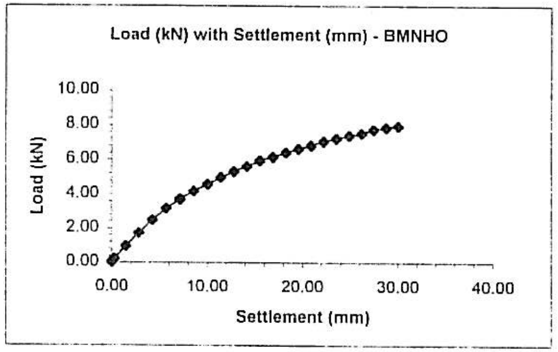

Load (kN) with Settlement (mm)- BMXH8D28
Load (kN)
Settlement (mm)

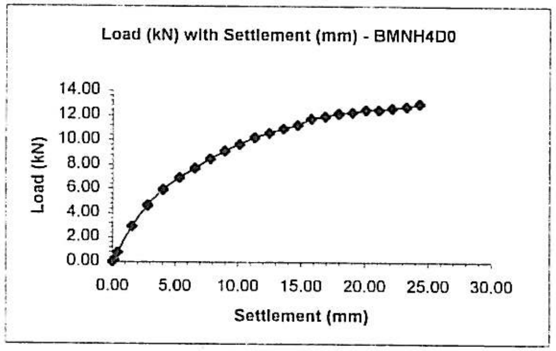
Model Testing

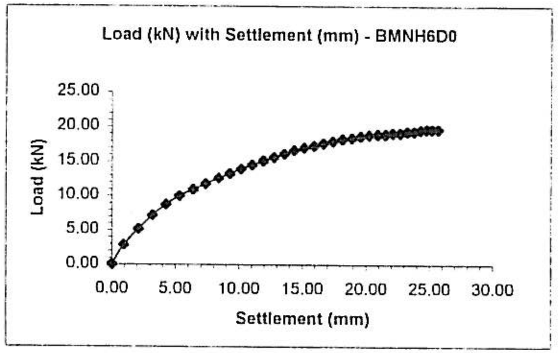
BMNH8D0

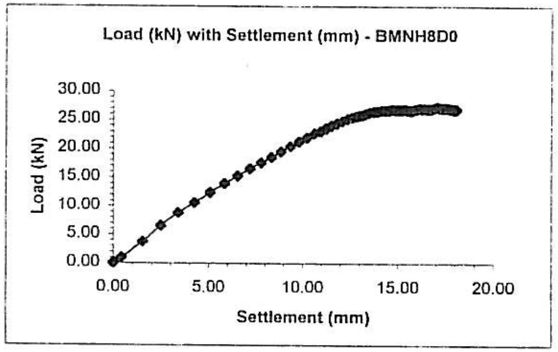
Load (kN)	Settlement (1) (mm)	Settlement (2) (mm)	Average (mm)
0.00	0.00	0.00	0.00
0.01	0.02	0.02	0.02
0.90	0.23	0.61	0.42
3.66	0.83	2.23	1.53
6.43	1.64	3.31	2.48
8.70	2.49	4.29	3.39
10.53	3.28	5.20	4.24
12.30	4.01	6.11	5.06
13.84	4.69	6.98	5.84
15.17	5.30	7.77	6.54
16.47	5.86	8.54	7.20
17.54	6.34	9.23	7.79
18.47	6.80	9.87	8.34
19.47	7.24	10.45	8.85
20.34	7.66	11.01	9.34
21.21	8.07	11.52	9.80
21.91	8.42	12.03	10.23
22.61	8.77	12.46	10.62
22.97	9.04	12.84	10.94
23.47	9.30	13.15	11.23
23.94	9.55	13.48	11.52
24.34	9.81	13.81	11.81
24.71	10.06	14.13	12.10
25.11	10.28	14.40	12.34
25.34	10.52	14.66	12.59
25.54	10.70	14.90	12.80
25.67	10.87	15.06	12.97
25.81	11.02	15.26	13.14
26.01	11.17	15.42	13.30
26.24	11.35	15.63	13.49
26.37	11.56	15.88	13.72
26.44	11.74	16.10	13.92
26.64	11.95	16.32	14.14
26.61	12.13	16.54	14.34
26.68	12.30	16.75	14.53
26.78	12.44	16.93	14.69
26.74	12.59	17.10	14.85
26.71	12.71	17.25	14.98
26.71	12.80	17.39	15.10
26.71	12.92	17.53	15.23
26.78	13.02	17.65	15.34
26.71	13.10	17.77	15.44
26.64	13.17	17.86	15.52
26.64	13.23	17.93	15.58
26.57	13.29	18.01	15.65
26.54	13.36	18.06	15.71
26.71	13.43	18.16	15.80
26.81	13.51	18.27	15.89
26.88	13.63	18.39	16.01
26.98	13.74	18.53	16.14
26.91	13.83	18.64	16.24
26.94	13.94	18.77	16.36
26.91	14.05	18.89	16.47
26.88	14.12	19.00	16.56
26.84	14.22	19.09	16.66
26.91	14.28	19.19	16.74
27.08	14.41	19.34	16.88
27.21	14.54	19.51	17.03
27.14	14.66	19.64	17.15
27.08	14.75	19.74	17.25
27.01	14.83	19.83	17.33
27.01	14.92	19.92	17.42
27.04	15.01	20.02	17.52
27.01	15.07	20.09	17.58
26.88	15.18	20.19	17.69
26.88	15.24	20.25	17.75
26.88	15.29	20.32	17.81
26.81	15.36	20.39	17.88
26.78	15.41	20.45	17.93
26.74	15.45	20.50	17.98
26.78	15.51	20.57	18.04

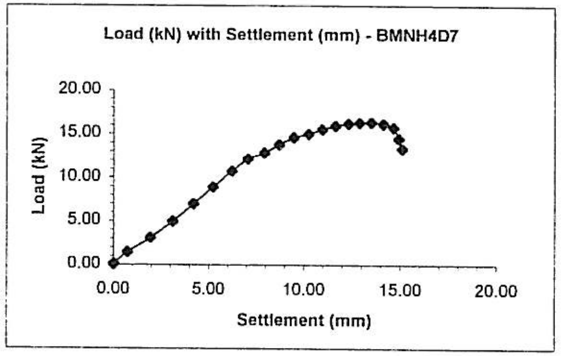


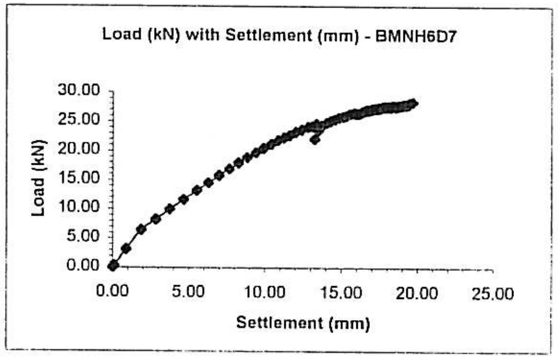

Load (kN) with Settlement (mm) - BMNHO
Load (kN)
10.00
8.00
6.00
4.00
2.00
0.00
0.00
10.00
20.00
30.00
40.00
Settlement (mm)

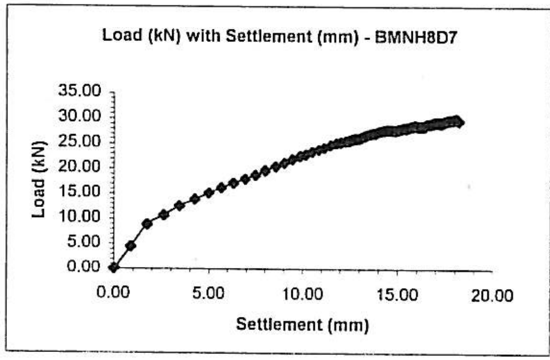

Load (kN) with Settlement (mm) - BMNH4D0
Load (kN)
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0.00
5.00
10.00
15.00
20.00
25.00
30.00
Settlement (mm)

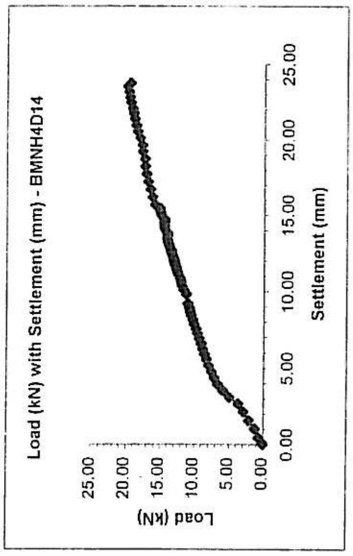

Load (kN) with Settlement (mm) - BMNH6D0
Load (kN)
25.00
20.00
15.00
10.00
5.00
0.00
0.00
5.00
10.00
15.00
20.00
25.00
30.00
Settlement (mm)

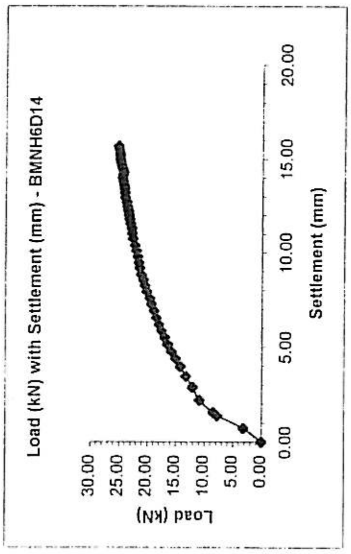

Load (kN) with Settlement (mm) - BMNH8D0
Load (kN)
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
5.00
10.00
15.00
20.00
Settlement (mm)

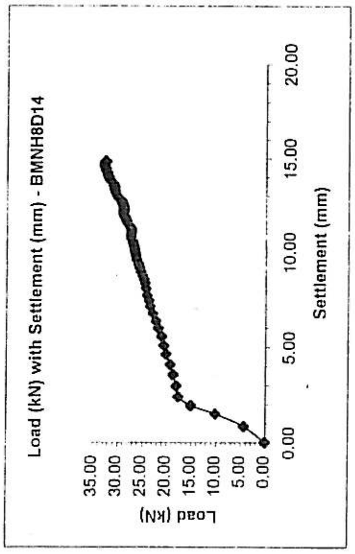

Load (kN) with Settlement (mm) - BMNH4D7
Load (kN)
20.00
15.00
10.00
5.00
0.00
0.00
5.00
10.00
15.00
20.00
Settlement (mm)

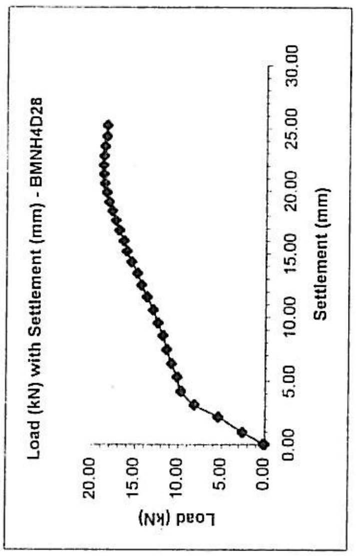

Load (kN) with Settlement (mm) - BMNH6D7
Load (kN)
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
5.00
10.00
15.00
20.00
25.00
Settlement (mm)

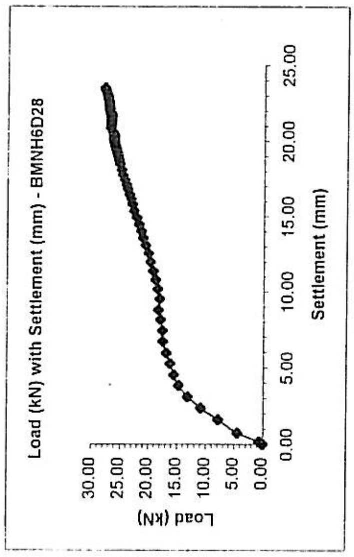

Load (kN) with Settlement (mm) - BMNH8D7
Load (kN)
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
0.00
5.00
10.00
15.00
20.00
Settlement (mm)

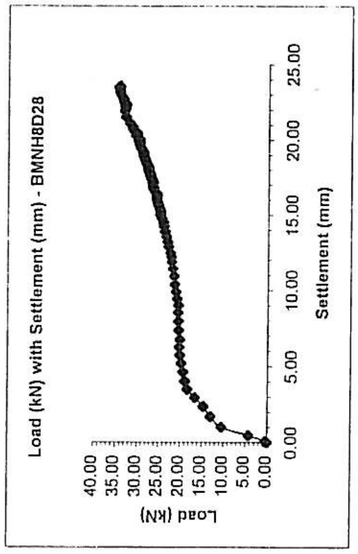

Load (kN) with Settlement (mm) - BMNH4D14
Load (kN)
Settlement (mm)
0.00 5.00 10.00 15.00 20.00 25.00


Load (kN) with Settlement (mm) - BMNH6D14
Load (kN)
Settlement (mm)
0.00 5.00 10.00 15.00 20.00 25.00 30.00
0.00 5.00 10.00 15.00 20.00


Load (kN) with Settlement (mm) - BMNH8D14
Load (kN)
Settlement (mm)
0.00 5.00 10.00 15.00 20.00 25.00 30.00 35.00
0.00 5.00 10.00 15.00 20.00


Load (kN) with Settlement (mm) - BMNH4D28
Load (kN)
Settlement (mm)
0.00 5.00 10.00 15.00 20.00
0.00 5.00 10.00 15.00 20.00 25.00 30.00


Load (kN) with Settlement (mm) - BMNH6D28
Load (kN)
Settlement (mm)
0.00 5.00 10.00 15.00 20.00 25.00 30.00
0.00 5.00 10.00 15.00 20.00 25.00


Load (kN) with Settlement (mm) - BMNH8D28
Load (kN)
Settlement (mm)
0.00 5.00 10.00 15.00 20.00 25.00 30.00 35.00 40.00
0.00 5.00 10.00 15.00 20.00 25.00

APPENDIX T

Publication

Khairul Anuar Kassim and Mustafa Kamal Shamshuddin (2001). ***Lime Stabilisation For Marginal Subgrade As Capping Layer For Road Construction.*** Proc. of the Seminar on Construction.Sept. 2001. Johor Bahru, Malaysia.